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Performance of Carbon Fibre Reinforced Polymer (CFRP) Strips for Flexural Strengthening of Reinforced Concrete Beams Made of High Strength Fly Ash Concrete

Submitted by

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For the fulfillment of the requirements for the degree of

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A THESIS
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Civil Engineering

BANDAR SERI ISKANDAR,
PERAK

JUNE, 2008

DECLARATION

I hereby declare that the thesis is based on my original work except for quotations and citations which have been duly acknowledged. I also declare that it has not been previously or concurrently submitted for any other degree at UTP or other institutions.

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ABSTRACT

Fibre reinforced polymer (FRP) composites have shown high potential as repair materials for civil infrastructures. The main attributes of FRP composites are high strength, light weight, good resistance to chemicals and non-magnetic and non-conductive properties. Compressive strength is one of the most important design parameters for concrete structures; it drives the design process and can influence the cost of a project. By the use of cement replacement materials (CRMs) at the optimum level the cost of concrete can be substantially reduced and its properties can also be enhanced. Fly ash (FA) is a by-product form coal-fired power plants and has extensively been used for producing high strength, high performance concrete. There is a lack of work in the CFRP external strengthening of reinforced concrete beams made of blended cement concrete.

In this study, three concrete mixes, with three different cementitious material contents such as 335, 360 and 380 kg/m^3 that incorporated fly ash content of 0%, 20%, 30% and 40% as the partial replacement of cement. The effects of CFRP system in terms of optimum length and ductility ratio for flexural strengthening were tested on 150 mm wide, 200 mm high and 2000 mm long RC beams. Three different lengths of 1333 mm, 1667 mm and 2000 mm of CFRP plate that represented 67%, 83% and 100% respectively of the total span length were chosen and the control beams were tested without CFRP strengthening. The behaviours of reinforced concrete beams made of normal and fly ash concretes strengthened with CFRP plates were compared and examined. It was found that high strength concrete such as 50 MPa can be produced by mixing as low as 228 kg/m^3 OPC content. Concrete compressive strength up to 84 MPa was obtained for the OPC content of 304 kg/m^3 with 20% fly ash content. It was found that by using CFRP system for external flexural strengthening, an increase in the load-carrying capacity up to 96% over the control beams was obtained. The length of the CFRP plate that achieved the optimum load-carrying capacity was 1667 mm which was 83% of the total span length. This length also provided an adequate ductility index of 3.1 compared to 4.6 for that of the control beam. The results of ultimate loads and deflections of normal and fly ash-based RC beams were obtained identical at first crack and at ultimate levels. It has been proved that the CFRP system could be used as effectively with fly ash based concrete beams as it has been used with normal concrete beams.

ABSTRAK

Pembaikpulihan di situ atau penguatan anggota konkrit bertetulang menggunakan plat keluli terikat telah terbukti sebagai kaedah efektif untuk membaiki prestasi struktur. Bagaimana pun terdapat beberapa kekurangan terwujud dalam penggunaan plat keluli terikat, sebagai contoh karatan dan kesulitan pengendalian dll. yang membuat penyelidik menerokai kemungkinan untuk menggunakan bahan polimer penetulangan gentian (PPG) untuk memberi sistem peningkatan kekutan yang tidak karat dan mudah. Kekuatan mampat adalah salah satu daripada parameter penting dalam rekabentuk untuk struktur konkrit; ia memandu proses rekabentuk dan boleh mempengaruhi kos projek. Dengan menggunakan bahan gantian simen (BGS) sebagai contoh abu terbang (AT) pada kadar optimum, kos konkrit boleh dikurangkan dengan banyak dan ciri-cirinya boleh dipertingakatkan. Dalam penyelidikan ini, tiga campuran konkrit dengan tiga kandungan bahan bersimen iaitu 335, 360, dan 380 kg/m^3 yang mengandungi kandungan abu terbang sebanyak 0%, 20%, 30% dan 40% sebagai bahan gantian simen telah dibina. Kesan sistem PPGK dari sudut panjang optimum dan nisbah kemuluran bagi penguatan lenturan diuji ke atas rasuk bertetulang bersaiz 150mm lebar, 200 mm tebal, dan 2000 mm panjang. Tiga panjang PPGK telah dipilih iaitu 1333 mm, 1667 mm, dan 2000 mm yang masing-masing mewakili 67%, 83%, dan 100% daripada jarak keseluruhan rentang. Kelakuan rasuk konkrit bertetulang (campuran normal dan dengan abu terbang) yang diperkuat dengan PPGK telah dibanding dan diteliti. Telah didapati bahawa konkrit kekuatan tinggi 50MPa boleh dihasilkan dengan mencampurkan kandungan simen Portland biasa serendah 228 kg/m^3 dengan 20% kandungan abu terbang. Kekuatan mampat konkrit setinggi 84MPa telah diperolehi untuk kandungan simen Portland biasa sebanyak 304 kg/m^3 dengan kandungan abu terbang 20%. Dengan menggunakan sistem PPGK bagi penguatan lenturan luaran, penambahan kapasiti menanggung beban sehingga 96% berbanding rasuk normal telah dicapai. Panjang plat PPGK yang menghasilkan kapasiti menanggung beban optimum ialah sepanjang 1667 mm iaitu 83% daripada jumlah panjang rentang. Panjang ini juga memberi indeks kemuluran purata yang memadai iaitu 3.1 berbanding 4.6 bagi rasuk kawalan. Perlakuan rasuk bertetulang yang telah diperkuat yang diperbuat dengan 30% AT dan konkrit normal didapati sama pada rekahan pertama dan pada takap muktamat. Ini membuktikan bahawa sistem PPGK

boleh digunakan secara efektif ke atas rasuk konkrit bertetulang yang mengandungi abu terbang dan simen Portland biasa.

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NOMENCLATURE

A_s	Cross-sectional area of tension steel reinforcement (mm^2).
A'_s	Cross-sectional area of compression steel reinforcement (mm^2).
A_f	Cross-sectional area of CFRP plate (mm^2).
$As_{min.}$	Minimum area of steel reinforcement (mm^2).
A_{sv}	Cross-sectional area of stirrups (mm).
a	Shear span (mm).
b	Width of rectangular cross section (mm).
b_f	Width of the CFRP plate (mm).
c	Height of the concrete compression block (mm).
C_c	Compression force in the concrete (N).
d	Effective depth of the beam section for tension reinforcement (mm).
d_c	Depth of compression reinforcement (mm).
d'	Depth of centroid of compression steel reinforcement from extreme compression fibre (mm).
d_f	Depth of FRP shear reinforcement (mm).
E_c	Modulus of elasticity of concrete (GPa).
E_f	Modulus of elasticity of CFRP plate (GPa).
E_s	Modulus of elasticity of steel (GPa).
f'_c	Cylinder's Compressive strength of concrete (MPa).
f_{cu}	Compressive strength of concrete (MPa).
f_s	Design service stress in the tension reinforcement (MPa).
f_y	Yield stress of steel reinforcement (MPa).
f_{yv}	Yield stress of stirrups (MPa).

gk	Dead load (KN/m^2).
h	Beam height (mm).
I_{cr}	Moment of inertia (mm^4).
k_m	Reduction factor for ultimate strain in the CFRP.
K	Constant, depend on the distribution of the bending moment in the member.
L	Effective or clear span length (mm).
L_{frp}	Length of the CFRP plate (mm).
M	Design moment (KN.m).
M_y	Yield moment (KN.m).
M_n	The nominal bending moment at failure (KN.m).
M_u	Ultimate moment (KN.m).
n	Number of CFRP layers.
qk	Imposed load (KN/m^2).
s_v	Spacing of the stirrups (mm).
t_f	Thickness of the CFRP plate (mm).
T_s	Tension force in the steel reinforcement (N).
T_f	Tension force in the CFRP reinforcement (N).
V	Design shear force (KN).
v_c	Design concrete shear stress (MPa).
v	Design shear stress (MPa).
z	Lever arm (mm).

GREEK SYMBOLS

ε_c	Concrete strain at extreme compression fibre at particular load.
ε_{cu}	Ultimate compressive strain of concrete.
ε_y	Yield strain of steel reinforcement.
ε_f	Strain of the CFRP plate at particular load.
ε_{fe}	Effective design strain for CFRP plate.
ε_{ff}	Strain of the CFRP plate at failure.
ε_{fu}	Ultimate strain of CFRP plate.
Δ_u	Mid-span deflection at ultimate load (mm)
Δ_y	Mid-span deflection at yield load (mm)
μ_D	Deflection ductility or Ductility index
ϕ	Mid-span curvature.
β_b	Percentage moment redistribution.
γ	Multiplier on f'_c to determine the intensity of an equivalent rectangular stress distribution for concrete.
Ψ_{frp}	Strength reduction factor for FRP.
σ	Stress in tensile steel at ultimate condition (MPa).
σ'	Stress in compressive steel at ultimate condition (MPa).

CHAPTER 1

INTRODUCTION

1.1 Background

Currently a large number of civil infrastructures are in a state of serious deterioration due to variety of reasons, among them aggressive ambient environmental effects are the most pronounced causes; these aggressive environmental effects could be corrosive due to carbonation and/or chloride attacks, alkali-silica reaction or any other form of chemical degradation, other reasons than the aggressive environmental effects would be the increase load specifications in the design codes. In order to maintain the efficiency and serviceability of older structures they must be rehabilitated so that they could meet the functional and other requirements of standards. To strengthen or rehabilitate the structure would be economically viable as a result of great deal of research done for developing cheaper repair techniques such as sprayed concrete, ferrocement, steel plate and fibre reinforced polymer (FRP) [Jumaat and Alam, (2006)].

Compressive strength of concrete is an important design parameter that controls the design process and can influence the cost of the project. Through effectively used cement replacement materials, the performance of concrete could be enhanced and its cost can be further reduced [Toutanji *et al.*, (2004)].

1.2 Application of Fibre Reinforced Polymer (FRP) in Reinforced Concrete

The ongoing efforts to upgrade, strengthen, retrofit, and/or rehabilitate the existing reinforced concrete (RC) structures, along with the development of advanced composite materials, led to the development of a new approach. This approach is based on application of externally bonded FRP strips to the concrete member. The method has shown many advantages, mainly due to the superior mechanical properties of the composite material and its applicability to broad range of structural members such as beams, columns, slabs, masonry, and walls. Various aspects of this innovative strengthening method have been investigated in the past few years. These aspects included the overall behavior of the strengthened beam; the response of strengthened precracked or pretensioned beams; shear strengthening of beams; the failure mechanisms of the strengthened member, which in many cases involved the sudden separation of the bonded FRP strip [Rabinovitch and Frostig, (2003)].

FRP composites are formed by embedding continuous fibres in a resin matrix which binds the fibres together. The common fibres are carbon fibres, glass fibres and aramid fibres [Teng *et al.*, (2001)]. Polymeric resins are used both as the matrix for the FRP and as the bonding adhesive between the FRP and concrete. All three types of FRP composites, namely; Glass Fibre Reinforced Polymer (GFRP), Carbon Fibre Reinforced Polymer (CFRP) and Aramid Fibre Reinforced Polymer (AFRP) have been used for strengthening of RC structures both in practical applications as well as in research activities.

1.3 Effectiveness of Cement Replacement Materials (CRMs) on Performance of Concrete

Since the last two decades, most of the efforts were made to explore the potential of various industrial by-products such as fly ash, slag and silica fume that can be used as cement replacement materials in order to improve the performance of concrete in terms of strength and durability. These materials have been advantageous in one way by decreasing the cost of concrete due to reduction in cement content; on the other hand they made it possible to produce high strength concrete that can manage to sustain the harsh exposure conditions.

1.4 Problem Statement

Since last two decades much efforts were made to produce high strength high performance concrete in order to avoid the large column sizes in tall buildings and long span bridges, which is required as a result of increased column loads. Most of the concrete mix designs for 28-day strength of 50 MPa or more as mentioned in the literature are based on higher amount of cement content such as 450 kg/m^3 or more. There is a great need of research work in order to produce a sufficiently high strength concrete at an optimum cement content such as less than 400 kg/m^3. Where the lower cement content will reduce the cost of concrete, on the other hand it will reduce the creep, shrinkage and other forms of distresses. High strength concrete in most of the cases is made of blended cement and it is widely used in bridges, tall buildings and other infrastructures.

Currently more than 70% of deteriorating structures, which required major and minor retrofitting and repair are bridges. Fibre reinforced polymer, FRP have shown great potential to be used as a major repair material in order to improve or regain the lost structural capacity of the member.

Available literature showed that studies on FRP based strengthening of structural member were mostly focused on to the type of fibre, its configuration and content without giving any serious emphasis on the characteristic of concrete such as high strength concrete or concrete containing different cement replacement materials, similarly optimum length of CFRP plate for effective strengthening has not been fully addressed. The principal aim of this research was to identify and fill the possible gaps in the knowledge of FRP structural strengthening techniques by giving particular emphasis on different types of concrete such as high strength concrete containing fly ash as a cement replacement material.

1.5 Objectives of the Study

In order to achieve the aims of this research study as highlighted in the problem statement, following were the principal objectives:

1. To develop the concrete mix design(s) for high strength concrete using moderate cement content by incorporating the optimum amounts of fly ash as a partial replacement to cement.
2. To establish the flexural behaviour, in terms of ultimate load capacity and ductility ratio, of reinforced concrete beams strengthened with externally bonded CFRP strips having different lengths in order to determine the optimum length of CFRP plate.
3. To verify the experimentally determined flexural capacity of reinforced concrete beams with theoretically calculated capacity of the beams using well recognized design models.

1.6 Scope of Study

For the purpose of achieving the desired objectives of the proposed research study, following scope of work was set:

1. To develop the high strength concrete mix: trial concrete mixes with different fly ash content were investigated by measuring the compressive strength at the ages of 3, 7, 28 and 90 days.
2. To determine the stiffness of structural component: modulus of elasticity, Ec of different concrete was determined using cylinder test at the ages of 28 and 90 days.
3. To determine the effects of CFRP strengthening on high strength concrete containing fly ash and to determine the optimum length of CFRP plate, 2 m long and 150 × 200 mm size concrete beams with 3 different lengths were tested under bending.

1.7 Organization of Thesis

The present research work investigates, experimentally and analytically, the flexural performance of reinforced concrete beams made of normal and fly ash concrete and strengthened with externally bonded CFRP plates.

A literature review on methods of strengthening, specially steel plate bonding and FRP composite bonding, effects of CFRP plates as strengthening material, and the effects of blended cement on properties of concrete, and the effects of using the CFRP for strengthening the concrete beams made of fly ash is presented in chapter 2.

Chapter 3 describes the main experimental program on mix design for high strength concrete (50 MPa) that incorporated different fly ash replacements (20%, 30% and 40% by weight) and the modulus of elasticity of those mixes. The methods adopted in the experimental phase of the study, along with a description of the material properties, the specimen geometry, fabrication, strengthening and testing is reported.

In Chapter 4, the main results and observations on 28-days compressive strength for blended cement concrete and load-deflection behaviour, ductility and ultimate loads of the tested beams, and a comparison of the experimental results with the theoretical results calculated from published data and ACI code are discussed. The effects of fly ash inclusion in RC strengthened with CFRP plates are also presented.

Chapter 5 gives a summary of the current study, along with the conclusions and the recommendations for future work.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Earlier codes of practices and specifications assumed that the concrete is very strong and long lasting material that can offer protection to the structure as a whole or to individual components against various kinds of attacks. However, there were a number of structures found inadequate for function much earlier than the anticipated service life. This kind of structures requires repair, maintenance and rehabilitation. The inadequacy of a structure or component may be due to mechanical damage, functional changes, overstress due to temperature changes, or corrosion of reinforcement. A common feature of a number of different causes of deterioration is that there is a reduction of the alkalinity of the concrete, which allows oxidation of the reinforcing steel to take place. This oxidation process leads to cracking of the concrete and possible spalling of the cover to the reinforcement [Thanoon *et al.*, 2005]. This chapter presents the detailed review of literature, which is very important for formulating problem statement, objective, scope and methodology. This review was mainly focused on methods of strengthening, specially steel plate bonding and FRP composite bonding, effects of CFRP plates as strengthening material, and the effects of cement replacements materials (CRMs) on properties of concrete, specially fly ash and the effects of using the CFRP for strengthening the concrete beams made of fly ash.

2.2 Methods of Strengthening and Retrofitting

There are many techniques available for strengthening and retrofitting of different reinforced concrete structural elements that include epoxy injection and cement grouting (which are widely used to treat cracking problem in concrete), ferrocement cover, section enlargement, steel plate bonding and FRP composite plate bonding, etc. Steel plate bonding technique, which is quite similar to the FRP bonding, can be compared with respect to common aspects of the FRP bonding technique.

2.2.1 Steel Plate Bonding

Much of the research studies to assess the performance of steel plate bonding technique were conducted simultaneously in South Africa and France in 1960s. The use of bonded external reinforcement was applied in UK in 1975 for the strengthening of the Quinton Bridges on the M5 motorway as a result of preliminary studies conducted by Irwin (1975), cited by Hollaway and Leeming, (1999).

Macdonald (1978) reported four-point loading tests on steel plated RC beams of length 4900 mm, these beams were used to provide data for the proposed strengthening of the Quinton Bridges, that incorporated two different types of epoxy adhesives, plate thicknesses of 10 mm and 6.5 mm giving width-to-thickness (b/t) ratios of 14 and 22 respectively. In all of the cases it was found that failure of the beams occurred at one of the ends due to horizontal shear in concrete adjacent to the steel plate, initiated at the plate end and resulted in sudden separation of the plate with the concrete that extended to the mid-span, cited by Hollaway and Leeming, (1999).

The external plate was found to have more significant effect in terms of crack control and stiffness. The load required to cause a crack width of 0.1 mm was increased by 95%, whilst the deflection under this load was substantially reduced. The post cracking stiffness was increased between 35% and 105% depending upon the type of adhesive used and the plate dimensions. The features of this work became the subject of more detailed programme of research at the Transport and Road Research Laboratory (TRRL), in which a series of RC beams of length 3500 mm were tested under four-point bending. The beams were plated as-cast or plated after being loaded to produce a maximum crack width of 0.1 mm. It was concluded that the plated as-cast and the precracked beams gave similar load-deflection curves, demonstrating the effectiveness of external plating for strengthening purposes [Hollaway and Leeming, 1999].

An extensive programme of research work conducted at the University of Sheffield since late 1970s, highlighted a number of effects of epoxy-bonded external steel plates on the serviceability and ultimate load behaviour of RC beams. Strengthening of existing structures using steel plates has also been investigated in Switzerland at the Swiss Federal Laboratories for Material Testing and Research (EMPA). Bending tests were performed on 3700 mm long RC beams, the plate width-to-thickness (b/t) ratio studied as the main variable while maintaining the plate cross-sectional area as constant. The results clearly showed that thin plating was more effective than that of thick narrow plating, alike studies as conducted in UK [Hollaway and Leeming, 1999].

Hussain *et al.*, (1995) investigated the use of anchor bolts at the ends of steel plated beams in an attempt to prevent brittle separation of the plate. In agreement with Jones *et al.* (1988) the bolts were 15 mm in diameter and penetrated to half the depth of the beam, they were found to improve the ductility of the plated beams considerably, but had only a marginal effect on the ultimate load capacity. The percentage improvement in ductility due to the addition of bolts was found to decrease as the plate thickness increased. It has been realized that in providing anchorage to the steel plated beams, considerable extra site work would be involved and this in turn could considerably increase the cost of the plate bonding technique, cited by Hollaway and Leeming, (1999).

Altin *et al.*, (2005) tested eleven reinforced concrete T-beams under monotonic loading in order to investigate the efficiency of epoxy-bonded steel plates for improving their shear capacity. One of the beams was chosen as a control and the other ten were strengthened in shear with different types of steel members subjected to different arrangements. The beam dimensions were 360 mm deep, 360 mm wide and 4000 mm long. All externally bonded steel plates had improved beam strength, stiffness and ductility. They also found that the type of steel member and its arrangement on the beam were among the effective parameters directing the ductility behaviour and determining the failure mode.

Barnes *et al.*, (2001) tested 9 reinforced concrete beams, of which 7 were strengthened with mild steel plates. Two strengthening techniques i.e. adhesive bonding and bolting were adopted. The beam dimensions were 175 mm wide, 400 mm deep and 2370 mm long. Adhesively bonded plates provided very high degree of surface crack control but inadequate surface area led to interface failure and sudden collapse. The bolted arrangement provided adequate plate anchorage up to the ultimate capacity of the section that allowed full utilization of the plates.

2.2.1.1 Disadvantages of External Strengthening Using Steel Plates

In-situ rehabilitation or upgrading of RC beams using bonded steel plates has proved to control flexural deformations and crack widths under service load, and to increase the load-carrying capacity of the member under ultimate conditions. It has been recognized as an effective, convenient and economical method for improvement of structural performance. Although the technique has shown success in practice, however, it also has disadvantages. Since the plates are not protected by the concrete in the same manner as the internal reinforcement, there exists the possibility of corrosion to occur, which could adversely affect the bond strength, hence could lead to failure of the strengthening system. To minimize the possibility of corrosion, all chloride-contaminated concrete should be removed prior to bonding and the plates must be treated with careful surface preparation. Integrity of the primer must be periodically checked, that cause a further maintenance task to the structure. In addition, the weight of the plates makes them difficult to transport and handle on site, particularly in areas of limited access, and can cause the dead weight of the structure to be increased significantly after installation. Elaborative and expensive falsework is required to maintain the plates in position during bonding [Hollaway and Leeming, 1999].

2.2.2 FRP Composite Bonding

In recent years, very extensive research has been conducted on the use of fibre-reinforced polymer (also referred to as fibre-reinforced plastic), FRP plates and/or sheets to replace steel plates for strengthening of structural members. The FRP plate bonding technology was first investigated in 1984 at the Swiss Federal Laboratory for Materials Testing and Research. Principal advantages of FRP plates are their high strength-to-weight ratio and great resistance to corrosion. The former property leads to great ease on site handling, reduces labour cost and interruptions to existing services, while the latter ensured durable performance. Normally the tensile strength of FRP plates is at least twice the strength of that of the steel plates, whereas their unit weight is just 20% that of steel. The common fibres are carbon fibres, glass fibres and aramid fibres [Teng *et al.*, 2001]. The composite strips have several advantages which are given below [Hollaway and Leeming, 1999]:

1. FRP composite plates can be manufactured with certain composition in order to meet the particular purpose and may comprise varying proportions of different fibres. The ultimate strength of the plates can thus be varied, but for strengthening schemes the ultimate strength of the plates is likely to be at least three times the ultimate strength of steel.
2. Density of FRP composite plates is only 20% of the density of the steel. Apart from transportation costs, the huge saving arises due to this is during installation. Composite plates do not require extensive jacking and support systems to move and hold in place. The adhesives alone will support the plate until curing has taken place. In contrast, fixing of steel plates constitutes a significant proportion of the work cost.
3. Due to light weight of FRP plates that a 20 m long plate may be carried in site by a single man. Some plates may also be bent into a coil as small as 1.5 m diameter and thus may be transported in a car or van without the need for lorries or subsequent craneage facilities.
4. Versatile design of systems: steel plates are limited in length due to their weight and handling difficulties. In-situ welding is not possible because it may cause damage to adhesives and expensive fixing of lap plates is therefore required. In contrast composite plates are of unlimited length, may be fixed in layers to suit strengthening requirements, and are thin enough that fixing in two directions can be accommodated by varying the adhesive thickness.
5. Easy and reliable surface preparation: steel plates require preparation by grit blasting, followed by careful protection until shortly before installation. In contrast composite plates may be produced with a peel-ply protective layer that may be easily stripped off just before the adhesive is applied.
6. Reduced mechanical fixing: composite plates are much thinner than the steel plates of equivalent capacity; this reduces peeling effects at the ends of the plates and thus reduces the likelihood of a need for end fixing. The overall depth of the

strengthening scheme is reduced; increase the headroom and improve the appearance.

7. Durability of strengthening system: there is the possibility of corrosion on the bonded face of steel plates, particularly if the concrete to which they are fixed is cracked or chloride contaminated. This could reduce the long term bond. Composite plates do not suffer from such deterioration.
8. Improved fire resistance: composite plates are a low conductor of heat as compared to steel, thus reduces the effect of fire on the underlying adhesives. The composite itself chars rather than burns and the system thus remains effective for a much longer period than the steel plate bonding.
9. Maintenance of strengthening system: steel plates require frequent maintenance that may cause traffic disruption and access cost as well as the work cost. Where as composite plates will not require such maintenance.
10. Reduced construction period: composite plates can be installed within very short time periods when compared with time taken for installation of steel plates.

Apart from the advantages of FRP composites, the disadvantages are:

1. Cost of plates: fibre reinforced composite plates are relatively more expensive than the equivalent capacity steel plates. However, the difference between the two materials is likely reduced as production volumes and competition between the manufacturers increases. Comparison of total contract costs for alternative methods of strengthening will be based on labour and access costs as well as material costs. Open competition has already shown that FRP composite plate bonding is the most economic solution in virtually all tested cases, without taking into account additional advantages such as durability.
2. Mechanical damage: FRP composite plates are more susceptible to damage than steel plates and could be damaged by a determined attack, such as with an axe. In

vulnerable areas with public access, the risk may be removed by covering the plate bonding with a render coat. Fortunately, if damage should occur to exposed FRP composite plate, such as by a high load, repairs can be undertaken much more easily than with a steel plate. A steel plate may be dislodged, or bond broken over a large area, which would damage bolt fixings and necessitate complete removal and replacement. However, with FRP composite plate bonding the damage is more likely to be localized, as the plate is thinner and more flexible, and the plate may be cut out over the damaged length and a new plate bonded over the top with an appropriate lap.

Depending on the fibres used, FRP composites are classified into three types: glass fibre reinforced polymer (GFRP) composites, aramid fibre reinforced polymer (AFRP) composites and carbon fibre reinforced polymer (CFRP) composites [Teng *et al.*, 2001]. The mechanical properties of such fibres are shown in Table 2.1.

Table 2.1 Typical mechanical properties of GFRP, CFRP and AFRP composites [Teng *et al.*, 2001]

Unidirectional advanced composite materials	Fibre content (% by weight)	Density (kg/m^3)	Longitudinal tensile modulus (GPa)	Tensile strength (MPa)
Glass fibre/ polyester GFRP laminate	50-80	1600-2000	20-55	400-1800
Carbon/epoxy CFRP laminate	65-75	1600-1900	120-250	1200-2250
Aramid/epoxy AFRP laminate	60-70	1050-1250	40-125	1000-1800

2.2.2.1 Glass Fibres Composites (GFRP)

Mukhopadhyaya *et al.*, (1998) tested six simply supported reinforced concrete beams, 150 x 250 x 3000 mm in size, in order to optimize the structural performance of beams strengthened with GFRP laminates. Effects of steel plate bonding and GFRP plate bonding were compared. An increase in the ultimate strength of about 15% was observed for beams strengthened with GFRP as compared to those strengthened with steel.

Kachlakev and McCurry, (2000) tested four full-scale reinforced concrete beams with similar geometry and placement of steel as applied in the Horsetail Creek Bridge (Columbia Gorge, Oregon) beams. The beams dimensions were 305 mm wide; 762 mm deep and 6096 mm length. The concrete strength was 20.7 MPa at the age of 28 days. One of the four beams was chosen as control beam with bottom flexural steel only without stirrups. The second, third and fourth beams were strengthened with flexural CFRP reinforcing, shear GFRP reinforcing and flexural CFRP and shear GFRP reinforcing respectively. The addition of GFRP for shear was sufficient to offset the lack of the stirrups and cause conventional RC beam failure by steel yielding at the mid-span.

Almusallam *et al.*, (2002) investigated the flexural behaviour of RC beams with epoxy bonded GFRP sheets. Twelve reinforced concrete beams (150 mm x 200 mm in cross section and 2050 mm long) were tested under four-point bending. The compressive strength of the concrete was in the range of 33-36.6 MPa. The main variables were the number of layers of GFRP sheets and the effect of precracking. An increase in the flexural strength from 16-32.3% was obtained. No difference between precracked and uncracked beams at the ultimate level was observed.

Xiong *et al.*, (2004) tested six reinforced concrete beams in order to investigate the behavoiur of beams externally strengthened with bonded hybrid carbon fibre-glass fibre sheets, the beams dimensions were 125 mm x 200 mm x 2300 mm and the concrete used had a compressive strength of 39.7 MPa. The first two beams were used as control samples, the next two were strengthened with the hybrid fibres that included two layers; the first layer was CFRP sheet of 0.11 mm thickness followed by the GFRP sheet of 0.167 mm thickness, and each of the last two beams was strengthened with a single and double layer of CFRP sheets respectively. The CFRP and GFRP sheet length adopted was the full-span length. The beams strengthened with the hybrid scheme CFRP and GFRP exhibited 90% and 101.5% higher load capacity with respect to the control beams respectively, and the last two beams exhibited 39% and 87% higher load capacity than the respective control beams.

2.2.2.2 Aramid Fibres Composites (AFRP)

Aramid fibres are usually very tough organic type synthetic fibres that generally characterized by having reasonably high tensile strengths up to 3000 MPa and moduli in the range of 60-120 GPa. It possesses very low density such as 1400 kg/m^3. Composites made with aramid fibres fit well into a gap in the range of stress/strain curves left by the family of carbon fibres at one extreme and glass fibres on the other. Aramid fibres are also fire-resistant and perform well at high temperatures. They are insulators of both electricity and heat and are resistant to organic solvent, fuel and lubricants. Some aramids have relatively very low compressive strength [Hollaway and Leeming, 1999].

In the tests performed by Chajes *et al.* (1995) they used glass, aramid and carbon fibre to strengthen concrete beams in shear. The important consideration was the orientation of the fabric fibres. For the beams tested, an increase in ultimate shear capacity of 60-150% was observed, cited by Björn Täljsten, (2003).

Ritchie *et al.*, (1991) studied the effectiveness of strengthening system using different types of FRP composites laminates. Laminates made of glass, carbon and aramid fibres have been used and the increase in ultimate strength was found to be in the range of 28 to 97% as compared with unstrengthened beams for different types of laminates, cited by Ramana *et al.*, (2000).

2.2.2.3 Carbon Fibres Composites (CFRP)

Carbon fibres are the most commonly used for strengthening to achieve high stiffness and high strength. The term carbon fibre (graphite fibres in the USA) covers a whole family of materials which encompass a large range of strengths and stiffness. The density of carbon fibre is 1900 kg/m^3, modulus of elasticity is in the range of 230-300 GPa, and tensile strength after processing is in the range of 3000-5000 MPa. Carbon fibre is most commonly produced from a precursor of polycrylonitrile (PAN) fibre; first it is processed by stretching it to achieve a high degree of molecular orientation, then it is stabilized in an oxidizing atmosphere while held under tension. The fibres are then subjected to a carbonizing regime at a temperature in the range of 1000-3500 $^{\circ}$C; the degree of carbonization determines the elastic modulus, density and electrical conductivity [Hollaway and Leeming, 1999].

2.3 CFRP Composites for Structural Strengthening

Among the various types of FRP, the application of carbon fibre reinforced polymer (CFRP) has received the most attention from the research community [Benjeddou *et al.*, 2007]. CFRP has shown high potential as repair material for bridge rehabilitation. The main attributes of CFRP composites are high strength, light weight, good resistance to chemicals and non-magnetic and non-conductive properties [Shahawy *et al.*, 1996].

2.3.1 Shear Strengthening Using CFRP Composites

Chaallal *et al.*, (1998) conducted an experimental investigation on the response of concrete beams strengthened in shear using externally applied epoxy bonded uni-directional CFRP strips. Results showed the feasibility of using this strengthening method to restore or increase the load-carrying capacity in shear of reinforced concrete beams while substantially reducing shear cracking.

Khalifa and Nanni, (2000) presented their study on the shear performance of six full-scale reinforced concrete RC T beams with that was 150 mm wide, 405 mm high and 3050 mm long and concrete compressive strength was 35 MPa. Different configurations of externally bonded carbon fibre-reinforced polymer (CFRP) sheets were used to strengthen the specimens in shear. The main parameters were CFRP amount and distribution (continuous sheets versus series of strips); bonded surface (two sides versus U-wrap). Test results indicated that the externally bonded CFRP reinforcement can be used to enhance the shear capacity of the beams; that increased the shear strength of 35-145%. CFRP applied on the beam sides gave 35% less shear contribution compared to U-wrap. The results also showed that there is optimum FRP quantity, beyond that the strengthening effectiveness is questionable.

Täljsten and Elfgren, (2000) tested eight full-scale reinforced concrete beams strengthened with CFRP-composites for shear. Two beams were preloaded up to failure and then strengthened with CFRP. The beams were 180 mm wide, 500 mm deep and 4500 mm long. Test results showed that the strengthening was very effective in shear with CFRP strips that were bonded to the face of the beams. A strengthening effect of almost 300% was achieved; it was even possible to reach a strengthening effect of 100% with completely fractured beams.

Khalifa and Nanni, (2002) tested twelve full-scale reinforced concrete beams externally strengthened with carbon fibre reinforced polymer (CFRP) sheets to study the shear behaviour and modes of failure. Beam dimensions were 150 mm x 305 mm in cross section and 3050 mm in length. Principal variables were shear stirrups and shear span-to-effective depth ratio as well as the amount and the distribution of CFRP. The concrete strength was of 19.3 and 27.5 MPa. The results revealed that the strengthening technique using CFRP sheets significantly increased the shear capacity; between 40-138%. The contribution of externally applied CFRP reinforcement to the shear capacity was influenced by the shear span-to-effective depth (a/d) ratio, i.e. increasing the a/d ratio increased the shear capacity.

Täljsten, (2003) conducted experimental investigation on the response of reinforced concrete beams strengthened in shear using CFRP sheets. Seven full-scale reinforced concrete beams of dimensions 180 x 500 mm and 4500 mm long were used. The concrete compressive strength was adopted between 58-71.4 MPa. The beams were strengthened with CFRP fabrics, which were consisted of 0°, 45° and 90° fabric angle. Results showed that if the beam was over-strengthened in shear the vertical placement of fabrics yielded good strengthening effect. An increase in shear capacity between 24-169 % was obtained.

Jayaprakash *et al.*, (2007) performed an experiment to investigate the shear capacity and failure modes of reinforced concrete rectangular beams. The main variables examined were the longitudinal tensile reinforcement ratio, shear span-to-effective depth ratio, spacing of CFRP strips and amount and orientation of CFRP strips. The concrete used to cast the beams was of grade 30 N/mm^2. Test results showed that the shear enhancement of CFRP strengthened beams varied between 11% and 139% with respect to the control beam. It was also concluded that by increasing the spacing of CFRP strips the strength of the strengthened beams decreased. Two failure modes, flexural and shear with CFRP rupture were observed at ultimate condition.

Bencardino *et al.*, (2007) presented the experimental results of four reinforced concrete beams made of concrete compressive strength of 40 MPa and without any internal shear reinforcement. Two beams were used as control beams without any external reinforcement and the other two beams were externally strengthened with a single CFRP laminate bonded to the tension face, one of the two beams was provided with external U-shaped steel stirrups. The results showed that bonding a CFRP laminate on the tension face of a reinforced concrete beam that was weak in shear, was not an adequate structural solution either to increase their load capacity or to change their mode of failure, the load capacity of the RC beam was not increased when compared to the control beam. When the CFRP laminate was externally anchored by a carefully designed system, the beam shear capacity was substantially increased to about 150% with a ductile mode of failure.

2.3.2 Flexural Strengthening Using CFRP Composites

Shahawy *et al.*, (1996) observed the behaviour of four reinforced concrete beams externally strengthened in flexure with bonded CFRP laminates. All beams were 203 mm wide, 305 mm deep and 2744 mm long. The width and thickness of the CFRP laminate were 300 mm and 0.171 mm respectively and the beams were strengthened over the full span length. The main variables were the number of CFRP layers. It has shown that the bending capacity can be increased considerably by bonding CFRP laminates to the tension side of beams, increases of 13, 66 and 92% for the beams strengthened with 1, 2 and 3 layers of CFRP laminates respectively as compared to the control beam were obtained. A reduction in deflection was also observed.

Norris *et al.*, (1997) conducted an experiment to study the effect of precracking of beams before strengthening with carbon fibre sheets. The significance of fibre direction was also examined. Nineteen beams were tested under four-point bending. They found that there was no difference in behaviour between the precracked beams and uncracked beams at the ultimate level, the most significant difference was observed due to the fibre orientations; when CFRP fibres were placed perpendicular to cracks in a test beam, a large increase in stiffness and flexural strength was observed and brittle failure occurred due to concrete rupture. When the CFRP fibres were placed obliquely to the cracks in the beam, a smaller increase in strength and stiffness was observed, however the mode of failure was ductile.

Hota *et al.*, (1998) performed an experiment to investigate the effect of wrapping of CFRP to the beam in flexure, to compare between steel-plate-reinforcement and carbon fibre reinforcement and to compare between the behaviour of damaged and undamaged beams when strengthened with CFRP sheet. Twenty four beams were tested under four-point bending. They found that the wrapped beams carried twice the load of unwrapped beams. Beams strengthened with carbon wrapping carried higher applied loads than steel-plate-reinforced beams. Damaged beams rehabilitated with carbon wrapping exhibited ultimate strength and stiffness performance similar to undamaged wrapped beams.

Naaman *et al.*, (1999, 2001) presented the results of tests on RC beams strengthened in flexure or shear with carbon FRP (either plates or sheets) and loaded under static or cyclic loads, at room or low temperatures. The test parameters were the amounts of reinforcing steel and FRP, concrete cover thickness and condition (with repair mortar used to simulate damaged concrete), and anchorage configurations. They found that, for a given reinforcement ratio, the ultimate load capacity increased but the ultimate deflection, and therefore ductility, decreased with the strengthening level. Also they obtained that extending the sheet to the supports, however, led to slightly higher ultimate loads and deflections, cited by Duthinh and Starnes, (2004).

Nguyen *et al.*, (2001) presented the experimental results of ten reinforced concrete beams of 120 x 150 mm cross section and 1500 mm length. The concrete compressive strength was in the range of 25-44.6 MPa. The main variables were the plate thickness, steel ratio and concrete cover thickness and their effects on the failure mode and the ultimate load of the strengthened beams. The plate width and thickness were 80 mm and 1.2 mm respectively. Four failure modes were observed in the strengthened beams namely: flexural failure, ripping of concrete, shear failure and a hybrid mode of shear failure and ripping, of which the last three modes were brittle. The beam that was bonded with the CFRP plate over its entire length failed in a flexural manner and exhibited an increase in the strength of about 167% as compared to the control beam. The addition of the CFRP increased the ultimate load by 26-236%.

A paper by Maalej and Bian, (2001) presented the results of an experimental investigation to study the effects of thickness of the CFRP plate on the load-carrying capacity. Five reinforced concrete beams were tested. The beam dimensions were 115 mm x 150 mm in cross section and 1500 mm in length. They found that increasing the thickness of CFRP plate increased the load-carrying capacity; the CFRP external reinforcement increased the beam load-carrying capacity by 22-46% and reduced the deflection capacities by 12-56% with respect to the control specimens.

Almusallam and AlSalloum, (2001) tested six reinforced concrete beams externally strengthened with carbon fibre reinforced polymer laminates. The beams were 150 mm wide, 200 mm deep and 2050 mm long. Concrete with compressive strength of 37.5 MPa was used. The beams in the first group were strengthened with one layer of FRP laminates while those in the second group were strengthened with two layers of FRP laminates; the thickness of the layer was 1.0 mm. The FRP laminate length was the full span. Test results indicated that the load carrying capacity of the beams strengthened with one and two layers of FRP laminates was increased by 131.9% and 192% respectively as compared to the control beam. The use of FRP laminates as an external reinforcement to strengthen and upgrade concrete structural members proved to be efficient and developed adequate ductility.

Hsu *et al*., (2003) performed an experiment in which twelve reinforced concrete beams were tested to failure under monotonic and cyclic loads. The main parameters were beam span, beam cross sections, steel ratios and with/without end anchorage. They found that bonding a CFRP strip to the tension face of the beam was an effective technique for repairing and retrofitting of reinforced concrete beams under both monotonic and cyclic loads.

David *et al*., (2003) performed an experiment to evaluate the effects of CFRP plates on reinforced concrete beams. Seven reinforced concrete beams of 150 mm wide, 300 mm height and 3000 mm long were tested in four-point bending and the CFRP plates were 1.2 mm thick and 50 mm width. The concrete compressive strength was 40 MPa. Externally bonded CFRP plates significantly increased the flexural strength of reinforced beams as compared to the control beam; an increase of about 50-75% was obtained.

A paper by Rabinovitch and Frostig, (2003) presented the test results of five rectangular reinforced concrete beams externally strengthened by CFRP bonded strips tested in four-point bending over clear span of 2100 mm (pre-loaded and non-pre-loaded). All beams were 200 mm wide, 200 mm deep and 2500 mm long. The concrete used had a compressive strength of 75 MPa. The CFRP strips were 120 mm wide 1.2 mm thick and the length was approximately full span. The investigation was focused on the stress concentrations that arise near the edge of the FRP strip and the brittle and sudden failure modes. The effect of the initial design of the RC beam was also examined. Two types of beams that differ in the longitudinal and shear reinforcement ratios were tested. The comparison of the preloaded and retrofitted beams with that of the upgraded beams (which were not pre-loaded) revealed that the ultimate loads of the rehabilitated beams were not significantly smaller than those of the upgraded beams. The results showed that the amount of the longitudinal reinforcement had almost no influence of the behaviour of the strengthened member. An increase in the ultimate load from 45-149% was obtained for the strengthened and repaired beams.

Ashour *et al.*, (2004) performed an experiment to investigate the flexural behaviour of reinforced concrete continuous beams externally strengthened with CFRP laminates; sixteen beams classified into three groups were cast, strengthened and tested. The position, length and thickness of the CFRP laminates were the main parameters; they found that using CFRP laminates for the strengthening of continuous beams was an effective technique and the load carrying capacity was increased. By increasing the thickness and length of the CFRP laminates the beam ductility was increased. Three failure modes of beams with external CFRP laminates were observed, namely laminate rupture, laminate separation and peeling failure of the concrete cover attached to the laminate. All the strengthened beams exhibited a higher load carrying capacity but lower ductility as compared to their respective control beams.

Gao *et al.*, (2004) tested eight reinforced concrete beams in four-point loading, the beam dimensions were 150 mm wide, 200 mm deep and 2000 mm long, the concrete compressive strength was 35.7 MPa. The CFRP plate width was 75 mm. Rubber modified resins were used for bonding the FRP strips to the beams. Both load-carrying capacity and ductility were increased due to usage of the rubber modified adhesive as compared to the control beams. An increase in the load carrying capacity from 27-63.8% was obtained.

Duthinh and Starnes, (2004) performed an experiment in which seven reinforced concrete beams of 150 mm width, 460 mm height and 2950 mm in length were tested. Five beams were strengthened with CFRP plates. The main parameters were the amount of internal reinforcement and CFRP. The application of CFRP plates was very effective for flexural strengthening of reinforced concrete beams.

Chahrour and Soudki, (2005) tested six reinforced concrete beams to investigate the flexural response of reinforced concrete beams strengthened with CFRP strips. The beams were 150 mm wide, 250 mm deep and 2400 mm long, the concrete compressive strength was 39 MPa. The observed mode of failure for the strengthened beams was the flexural shear crack-induced interfacial debonding type. Externally bonded CFRP strips significantly increased the ultimate capacity of reinforced concrete beams; an increase of 45% was obtained while the ductility was reduced.

A test performed by Maalej and Leong, (2005) in which seventeen reinforced concrete beams with different sizes were tested in flexure. The variables were the beam size and FRP thickness, normal strength concrete was used. They obtained that the size of beam did not significantly affect the extent to which a reinforced concrete beam can be strengthened.

Kotynia, (2005) performed an experiment to study the debonding failures of reinforced concrete beams strengthened with CFRP strips, three series of twenty beams were tested, their dimensions were 150 mm wide, 300 mm deep and the length was variable. Two failure modes were observed; plate-end debonding with concrete cover separation and mid-span debonding. He also found that by the strip extending closer to the beam support, the ultimate load was increased by 25%.

A test performed by Toutanji *et al.*, (2006) in which eight reinforced concrete beams were tested and analyzed: one control beam and seven beams reinforced with three to six layers of CFRP sheets bonded by inorganic epoxy. The test results showed that increasing the number of layers of CFRP sheets increased the load-carrying capacity but reduced the ductility of the strengthened beams. For three and four layers of reinforcement beams failed by the rupture of carbon fibre sheet, and for six layers of FRP reinforcement beams failed by FRP delamination.

Li *et al.*, (2006) tested seven reinforced concrete beams to investigate the effects of thickness and length of CFRP sheet on the failure mode, stiffness and ductility of reinforced concrete beams. All beams were 120 mm wide, 200 mm deep and 2000 mm long. The concrete compressive strength was in the range of 37-40 MPa. The main parameters were the CFRP sheet thickness and length. CFRP effectively increased the initial cracking loads, ultimate loads, stiffness and ductility of concrete beams. The results indicated that by increasing the thickness and length of CFRP sheet the stiffness of the strengthened beams was increased. Debonding failure had greater influence on initial cracking load than on stiffness, ductility and ultimate loads of concrete beams and had a lesser influence on crack pattern.

Wenwei and Guo, (2006) tested seven reinforced concrete beams. One beam was used as control beam and the other six were strengthened in flexure using two layers of CFRP laminates; all beams were 150 mm wide, 250 mm deep and 2700 mm long. The concrete used had a compressive strength of 30 MPa. The CFRP laminate width was 150 mm and the thickness of the layer was 0.111 mm and length of the laminate was approximately 96% of the span length. All specimens were tested under four-point bending over a 2400 mm simple span. The main parameters were different levels of sustaining load. CFRP external reinforcement increased the load carrying capacity of load-damaged beams by

22.5-41.2%, but strengthening with externally bonded CFRP laminates under sustaining loads reduced the ductility of the strengthened beams.

Esfahani *et al.*, (2007) performed an experiment on reinforced concrete beams externally strengthened with CFRP sheets with different reinforcement ratios. Twelve reinforced concrete beams, each of 150 mm width, 200 mm height and 2000 mm length were manufactured with concrete compressive strength of 25 MPa and tested in four-point bending. They obtained that with large reinforcement ratio close to the maximum reinforcement ratio, ρ_{max}, failure of the strengthened beams occurred in either interfacial debonding induced by flexural crack, or interfacial debonding induced by flexural-shear crack with adequate ductility.

Benjeddou *et al.*, (2007) tested eight reinforced concrete beams to investigate the behaviour of damaged reinforced concrete beams repaired by bonding of CFRP laminates. All beams were 120 mm wide, 150 mm deep and 2000 mm long. The main parameters were damage degree, CFRP laminate width and concrete strength class. The concrete compressive strength was in the range of 21-38 MPa. The CFRP laminate width was varied from 50 to 100 mm and the full span plate length was adopted. An increase in the load capacity for the beam that was directly strengthened (not damaged) of 87% as compared to the control beam was obtained. Two failure modes were observed for strengthened and repaired beams, namely, peeling off and interfacial debonding. The former was observed for all reinforced beams with the laminate width of 100 mm while the latter was observed for all reinforced beams with the laminate width of 50 mm.

2.3.3 Modes of Failure

From the number of papers reviewed and summarized in this section it can be concluded that typical failure modes of FRP strengthened beams can be grouped into seven categories. These are (i) flexural failure by FRP rupture, (ii) flexural failure by crushing of compressive concrete, (iii) shear failure, (iv) concrete cover separation, (v) plate end interfacial debonding, (vi) interfacial debonding induced by intermediate flexural cracks and (vii) interfacial debonding induced by intermediate flexural shear cracks. Concrete cover separation and various types of interfacial debonding are premature failure modes, which can prevent the full utilization of the tensile strength of the FRP plate.

A summary of the review on fibre reinforced polymer plates and its applications for external strengthening of reinforced concrete beams is given in Table 2.2.

Table 2.2 Summary of the literature review on strengthening of RC beams by bonding FRP composites

No.	Researcher(s)	Year	Beam dimensions (mm) b x h x L	Type of concrete used	Concrete compressive strength (MPa)	Type of FRP composite	The investigated parameters	Number of layers of CFRP	Type of strengthening	Increase in load capacity (%)
1	Shahawy *et al.*	1996	203x305x2744	Normal	29 and 41	CFRP	Number of CFRP layers	1, 2, 3	Flexural	13, 66, 92
2	Kachlakev and McCurry	2000	305x762x6096	Normal	20.7	CFRP and GFRP	Configuration and type of FRP composite	1	Flexural and shear	150
3	Khalifa and Nanni	2000	150x405x3050 (T-beam)	Normal	35	CFRP	Configuration of CFRP	n/a	Shear	35-145
4	Täljsten and Elfgren	2000	180x500x4500	Normal	48-65.2	CFRP	Effect of precracking	n/a	shear	100-300
5	Maalej and Bian	2001	115x150x1500	Normal	43.3	CFRP	Effect of thickness of CFRP plate	1, 2, 3, 4	Flexural	22-46
6	Almusallam and Salloum	2001	150x200x2050	Normal	37.5	CFRP and GFRP	Number of layers and effect of precracking	1, 2	Flexural	132-200

n/a: Not available

Table 2.2 Summary of the literature review on strengthening of RC beams by bonding FRP composites (continued)

No.	Researcher(s)	Year	Beam dimensions (mm) b x h x L	Type of concrete used	Concrete compressive strength (MPa)	Type of FRP composite	The investigated parameters	Number of layers of CFRP	Type of strengthening	Increase in load capacity (%)
7	Khalifa and Nanni	2002	150x305x3050	Normal	19.3 and 27.5	CFRP	Steel stirrups, a/d ratio, amount and distribution of CFRP	1, 2	Shear	40-138
8	Täljsten	2003	180x500x4500	Normal	58-71.4	CFRP	Configuration and direction of CFRP sheets	n/a	Shear	24-169
9	David *et al.*	2003	150x300x2800	Normal	40	CFRP	Number of layers and effect of precracking	1, 2	Flexural	50-75
10	Rabinovitch and Frostig	2003	200x200x2500	Normal	47-75	CFRP	Shear reinforcement, plate edge configuration and effect of preloading	1	Flexural	45-149
11	Gao *et al.*	2004	150x200x2000	Normal	35.7	CFRP	Thickness of CFRP strip	1	Flexural	27-63.8
12	Almusallam and Salloum	2002	150x200x2050	Normal	33-36.6	GFRP	Number of layers and effect of precracking	1, 2	Flexural	16-32.3

n/a: Not available

Table 2.2 Summary of the literature review on strengthening of RC beams by bonding FRP composites (continued)

No.	Researcher(s)	Year	Beam dimensions (mm) b x h x L	Type of concrete used	Concrete compressive strength (MPa)	Type of FRP composite	The investigated parameters	Number of layers of CFRP	Type of strengthening	Increase in load capacity (%)
13	Chahrour and Soudki	2005	150x250x2400	Normal	39.0	CFRP	Length of CFRP strip	1	Flexural	24-45.1
14	Wenwei and Guo	2006	150x250x2700	Normal	30	CFRP	Level of sustaining load	2	Flexural	22.5-41.2
15	Esfahani *et al.*	2007	150x200x2000	Normal	25	CFRP	Effect of reinforcing bar ratio	1-3	Flexural	12-56.5
16	Jayaprakash *et al.*	2007	120x310x2980	Normal	30	CFRP	Tension reinforcement ratio, a/d ratio and spacing, amount and orientation of CFRP strips	n/a	Shear	11-139
17	Benjeddou *et al.*	2007	120x150x2000	Normal	21 and 38	CFRP	Damage degree, CFRP laminate width and concrete strength class	1	Flexural	40-87

n/a: Not available

2.4 Effects of Cement Replacement Materials (CRMs) on Concrete Properties

In recent years, high-strength concrete has increasingly been used in civil engineering structures because of various advantages such as reducing the sizes of beams and columns, which are essential in high-rise building [Jaturapitkkul *et al.*, (2004)]. Mineral admixtures such as silica fume (SF), ground granulated blast-furnace slag (GGBS) and fly ash (FA) have shown much improvement in engineering properties and performance of concrete when they are used as mineral additives or as partial replacement to cement [Oner *et al.*, (2005)].

2.4.1 Silica Fume (SF)

Silica fume is also referred as microsilica or condensed silica fume, but the term 'silica fume' has become generally accepted. It is a by-product of the manufacturing of silicon and ferrosilicon alloys from high-purity quartz and coal in a submerged-arc electric furnace. It has a high content of glassy silicon dioxide (SiO_2); hence, the term given silica fume. Silica in the form of glass (amorphous) is highly reactive, and the tiny size of the particles speeds up the reaction with calcium hydroxide evolved due to the hydration of Portland cement. The very small particles of silica fume can enter into the spaces between the particles of cement, and thus improve packing. When the furnace has an efficient heat recovery system, most of the carbon is burnt so that silica fume is virtually free from carbon and is light in colour. Furnaces without a full heat recovery system leave some carbon in the fume, which is therefore dark in colour. It can be added that silica fume is expensive [Neville, (1999)]. Silica fume has been used for producing grade 80 concrete for construction of highly loaded lower columns of PETRONAS twin towers.

Goldman and Bentur (1989), Huang and Feldman (1985), and Toutanji and El-Korchi (1995) claim that silica fume improves the strength of the bond between the aggregates and the cement matrix. The partial replacement of cement by silica fume increases the strength of mortar and concrete; yet it does not seem to have an important effect on the strength of pure cement paste. To other researchers, however, the positive result due to the admixture of silica fume stems from the increase in strength of the cement matrix. Many researchers are not in agreement about the definition of the optimal content of silica fume, which entails the development of high strength. To some researchers, the content is about 15%, whereas to others, the increase in compressive strength may reach from 30 to 40% of replacement of cement by silica fume, cited by Duval and Kadri, (1998).

Duval and Kadri, (1998), replaced the cement with 0%, 10%, 20% and 30% silica fume while investigating the influence of silica fume on the workability and the compressive strength of high-performance concrete. The cement content was varied from 310-550 kg/m^3, water/cementitious materials ratio was varied between 0.25-0.45 and the superplasticizer was used in the range of 0.4-5.5% of the binder content. They obtained that the increase of the compressive strength of silica fume concretes much depends on the decrease of the water/cementitious materials ratio than on the replacement of silica fume with cement. The compressive strength increased up to 20% and reaches a maximum for 10 to 15% silica fume level. However the gain in strength compared with reference concrete remained less than 15%.

2.4.2 Ground Granulated Blast-Furnace Slag (GGBS)

The slag is a waste product in the manufacture of pig iron, about 300 kg of slag being produced for each tonne of pig iron. Chemically, slag is a mixture of lime, silica and alumina. Blast-furnace slag varies greatly in composition and physical structure depending on the processes used and on the method of cooling of the slag [Neville, (1999)].

GGBS is a by-product of the iron industry and when combined properly with Portland cement in concrete, GGBS can improve the hardened properties of concrete. GGBS rises to the top of molten iron in the blast-furnace, from there it is separated from the iron, quenched with water and ground to a fineness that is similar to Portland cement. GGBS is composed mainly of lime, silica and alumina. When blended with Portland cement, GGBS can improve the fresh and hardened properties of concrete. Appropriate additions of GGBS can increase compressive strength, reduce permeability and increase sulfate resistance [Malhotra, (2004)].

Oner and Akyuz, (2007) replaced the cement with 0%, 15%, 30%, 50%, 70%, 90% GGBS for their investigation on the optimum usage of GGBS for the compressive strength of concrete. 32 concrete mixes were prepared in four groups according to the binder content. The binder content was in the range of 250-400 kg/m^3. The early age strength of GGBS concretes was lower than the control concretes with the same binder content. However, as the curing period was extended, the strength increase was higher for the GGBS concretes. The reason is that, the pozzolanic reaction is slow and the formation of calcium hydroxide requires time. The compressive strength of GGBS concrete increased as the GGBS content was increased up to an optimum point, after which the compressive strength decreased.

2.4.3 Fly Ash (FA)

Fly ash, also known as pulverized-fuel ash, is the ash precipitated electrostatically or mechanically from the exhaust gases of coal-fired power stations; it is the most common artificial pozzolana. The fly ash particles are spherical which is advantageous from the water requirement point of view and have a very high fineness: the vast majority of particles have a diameter between less than 1 μm and 100 μm, and specific surface of fly ash is usually between 250 and 600 m^2/kg. The high specific surface of the fly ash means that the material is readily available for reaction with calcium hydroxide. The American classification of fly ash is based on the type of coal from which the ash originates. The most common fly ash derives from bituminous coal, is mainly siliceous, and is known as Class F fly ash. Sub-bituminous coal and lignite result in high-lime ash, known as Class C fly ash [Neville, (1999)].

2.4.3.1 Effects of Fly Ash on Properties of Fresh Concrete

The main influence is that on water demand and on workability. For a constant workability, the reduction in the water demand of concrete due to fly ash is usually between 5 and 15% by comparison with a Portland-cement-only mix having the same cementitious material content. A concrete mix containing fly ash is cohesive and has a reduced bleeding capacity. The mix can be suitable for pumping and for slipforming; finishing operations of fly ash concrete are made easier. The influence of fly ash on the properties of fresh concrete is linked to the shape of fly ash particles. The reduction in water demand of concrete caused by the presence of fly ash is usually ascribed to their spherical shape; this is being called a 'ball-bearing effect' [Neville, (1999)].

2.4.3.2 Effects of Fly Ash on the Compressive Strength of Concrete

The reactions of fly ash are affected by the properties of Portland cement with which it is used. Moreover, in addition to the effect of chemical reactions, fly ash has a physical effect of improving the microstructure of the hydrated cement paste. The main physical action is that of packing of the fly ash particles at the interface of coarse aggregate particles [Neville, (1999)].

Ramezanianpour and Malhotra, (1995) replaced the ordinary Portland cement by 25% and 58% class F fly ash with water/cementitious material ratios of 0.5 and 0.35 respectively, the cement content was 372 kg/m^3. Superplasticizer was used with 1% by weight of cement only for the mix containing 25% replacement. Under moist curing the 28-day compressive strengths for 25 % and 58 % fly ash concrete were 31.5 MPa and 32.1 MPa respectively.

Malhotra and Hemmings, (1995) replaced the cement with fly ash up to 58% by weight of cement, with 1.1% superplasticizer. They reported 30 and 42 MPa average cylinder compressive strength at 28 and 91 days respectively. The water/cementitious material ratio of 0.32 and a total of 360 kg/m^3 cementitious material was used.

Papadakis, (1999) replaced the cement with 10%, 20% and 30% fly ash to investigate the effect of low calcium fly ash on Portland cement systems, the cementitious material content was of 514 kg/m^3 and the water/cementitious material ratio of the concrete reported was in the range of 0.56-0.72. The compressive strength was obtained as 45-60 MPa at 28 days and 60-70 MPa at 90 days.

Poon *et al.*, (2000) prepared two series of concrete mixtures with w/c ratios of 0.24 and 0.19 respectively. Fly ash was used in the proportions of 0%, 25% and 45% of the total cementitious material. All concrete mixtures had 637 kg/m^3 cementitious material content; commercially available Portland cement equivalent to ASTM type I was used. At the w/c of 0.24 the mix containing 25% fly ash showed slightly lower compressive strength at the ages of 3 and 7 days, but higher compressive strength at the ages of 28 and 90 days as compared to the control mix. The mix with 45% fly ash showed a 28-day compressive strength lower than the reference mix by about 8%. The compressive strengths obtained at 28 and 90 days were 95-113.3 MPa and 104-136.9 MPa respectively. However lowering w/c ratio to 0.19 did not further improved the concrete strength.

Jiang and Malhotra, (2000) replaced the cement with 55% fly ash by mass, the water/cementitious material ratio was in the range of 0.34 to 0.43. The cementitious material content was 400 kg/m^3; superplasticizer was used as a water-reducing admixture. They reported cylinder compressive strengths of concrete of 30.7-55.8 MPa at 28 days and 43.9-65.2 MPa at 90 days.

Jiang *et al.*, (2000) replaced cement with 70% fly ash by mass, water/cementitious material ratio was 0.37 and the cementitious material content was 333 kg/m^3. They reported concrete compressive strength of 26 MPa at 28 days and 36 MPa at 90 days.

Bouzoubaâ *et al.*, (2000, 2001) replaced the cement with fly ash up to 55-58% by weight of cement and utilized various amounts of superplasticizer to maintain the workability. The water/cementitious material ratio of the concrete reported was in the range of 0.33 and contained 370 kg/m^3 cementitious materials. They reported 30 and 42 MPa average cylinder compressive strength at 28 and 91 days respectively.

Bouzoubaâ and Lachemi, (2001) replaced the cement with 40%, 50% and 60% class F fly ash, the water/cementitious material ratio was in the range of 0.35 to 0.45 to obtain self-compacting concrete (SCC) with targeted 28-day compressive strength of 35 MPa. The cementitious material content was 400 kg/m^3. The SCC developed compressive strengths ranging from 15 to 31 MPa and from 26 to 48 MPa, at 7 and 28 days respectively.

Yin *et al.*, (2002) replaced the ordinary Portland cement with 30-50% fly ash while investigating on the compounding and application of C80-C100 high-performance concrete (HPC). The cementitious material content was 580 Kg/m^3. They reported 28-day compressive strength of 95.2 MPa at 0.23 water/cementitious material ratio.

Xie *et al.*, (2002) studied the preparation technology of high-strength self-compacting concrete (SCC) containing ultrapulverized fly ash (UPFA), main mix parameters were examined. When the UPFA replaced 30%, 40% and 50% cement, the total cementitious material was 540 kg/m^3, the water/cementitious material ratio was in the range of 0.27-0.3 and the corresponding 28-day compressive strengths were 71.1, 67.7 and 58.2 MPa respectively. When the UPFA replaced 30% cement, the total cementitious material was 693 kg/m^3, the 28-day compressive strength for SCC and Ordinary Concrete (OC) were 79.6 MPa and 84.2 MPa respectively.

Toutanji *et al.*, (2004) performed an experiment in which three mixes containing 20%, 25% and 30% fly ash as a cement replacement material were cast, cured for 14 days and tested. Type I ASTM cement was used; the w/c ratio was 0.4 and 1.5% superplasticizer was used. The addition of fly ash exhibited a reduction in compressive strength and it was reduced by 50% with the addition of 30% fly ash, this reduction was attributed to the fact that specimens were cured for a short period of time (only 14 days). Because of low pozzolanic reaction of fly ash, continuous wet curing and favourable curing temperatures are required for proper development of strength.

Jaturapitakkul *et al.*, (2004) used the ground coarse fly ash (reclassified) to replace the Portland cement type I with 0%, 15%, 25%, 35% and 50% by weight of cement. The cementitious material content in all the concrete mixes was 554 kg/m^3, the water/cementitious material ratio was 0.27. The strength development of concrete containing 15%, 25% and 35% ground coarse fly ash as cement replacement was faster than that of 50% cement replacement, while the 25% gave the highest compressive strength at all ages. The replacement of Portland cement type I with 15%, 25% and 35% and 50% of ground coarse fly ash produced high-strength concrete. The compressive strength of 50% replacement was almost the same as that of the control concretes.

Oner *et al.*, (2005) replaced the ordinary Portland cement with 15%, 25%, 33%, 42%, 50% and 58% class F fly ash by weight of cement. The cementitious material content was 320 kg/m^3 and the water/cementitious material ratio was in the range of 0.74-0.8. The obtained 28-day compressive strength was in the range of 39.3-42.7 MPa.

Berryman *et al.*, (2005) used various percentages of class C (25-65%) and class F (25-75%) fly ash and water-reducing admixture (WRA) under field manufacturing conditions. 2.0% WRA was used. 7-day compressive strength was found to be highest when the concrete mix included approximately 35% class C or 25% class F fly ash. High-percentage replacement of cement with fly ash was recommended only in situations where early compressive strength is not required.

McCarthy and Dhir, (2005) used the fly ash with 0%, 15%, 30% and 45% by mass of cement for design strength grades (25-70 N/mm^2), for 50 and 70 N/mm^2 the cement content was 385 and 510 kg/m^3 respectively. High fly ash levels could be used in combination with Portland cement to produce concrete covering the range of design strengths typically required in practice. Overcoming early strength shortfall and matching that of Portland cement concrete can be solved through the use of rapid hardening Portland cement. The fresh properties, including workability, drying shrinkage and durability of high volume fly ash concrete, indicated either similar or enhanced performance to Portland cement concrete.

Duran Atiş, (2005) performed a laboratory investigation to evaluate the strength properties of high-volume fly ash roller compacted and superplasticized workable concrete cured at moist and dry conditions. The normal Portland cement was replaced with two different low-lime class F fly ashes, good and low quality with 0%, 50% and 70%, the cement content was 400 kg/m^3. The concrete that contained 50% good quality fly ash developed high strength while 70% good quality fly ash replacement concrete developed moderate strength. The concrete that contained 50% low quality fly ash developed satisfactory strength at 28 days and high strength at 1 year.

Sata et al., (2007) used two types of fly ash (pulverized coal combustion with a burning temperature of 1300-1400 °C (FA) and fluidized bed combustion with a burning temperature of 800-900 °C (FB)) to partially replace Portland cement with 0%, 10%, 20%, 30% and 40% by weight. All concrete mixtures had 560 kg/m3 cementitious material content, water/cementitious material ratio was kept constant at 0.28. At the early age (7 days) the FA concretes yielded low compressive strengths than that of the control sample, after 28 days the compressive strengths of FA concretes tended to increase with the curing age for all replacements. At all curing ages, all FB concretes produced higher compressive strength than that of the control. The concrete compressive strength was in the range of 78-84 MPa and 86-92 MPa at 28 and 90 days respectively. The fly ash FA and FB with high fineness were very reactive pozzolanic materials and could be used in making high-strength concrete. The use of FA and FB in high-strength concrete had no significant effect on the modulus of elasticity of the concrete as compared to the control sample.

Shafiq *et al.*, (2007) replaced the ordinary Portland cement with 0%, 30% and 40% fly ash from Malaysian source, Manjung power station at Lumut, the cementitious material content was 325 kg/m^3 and the water/cementitious material ratio was in the range of 0.515-0.560. They obtained 28 and 90-day compressive strength in the range of 25-40 MPa and 35-40 MPa respectively. For a particular mix, after 90 days the fly ash concrete exhibited 50-52% increment in strength as compared to the 28-day strength for that mix.

A summary of the literature review on the effects of fly ash on hardened concrete properties is given in Table 2.3.

Table 2.3 Summary of the literature review on fly ash and its effect on the concrete compressive strength

No.	Researcher(s)	Year	Fly ash replacement (%)	Cement content (kg/m^3)	Water/binder ratio	Admixtures	28-day compressive strength (MPa)	90-day compressive strength (MPa)
1	Ramezanianpour and Malhotra	1995	25	372	0.5	SP	31.5	45-53
			58		0.35	SP	32.1	n/a
2	Malhotra and Hemmings	1995	58	360	0.32	SP	30	42
3	Haque and Kayali	1998	10	500	0.25-0.38	SP	111	n/a
			15				102	
4	Papadakis	1999	10, 20, 30	514	0.56-0.72	n/a	45-60	60-70
5	Poon *et al.*	2000	25	637	0.24 and 0.19	SP	95-113.3	104-136.9
			45					
6	Jiang and Malhotra	2000	55	400	0.34-0.43	SP	30.7-55.8	43.9-65.2

n/a: Not available

Table 2.3 Summary of the literature review on fly ash and its effect on the concrete compressive strength (continued)

No.	Researcher(s)	Year	Fly ash replacement (%)	Cement content (kg/m^3)	Water/binder ratio	Admixtures	28-day compressive strength (MPa)	90-day compressive strength (MPa)
7	Jiang *et al.*	2000	70	333	0.37	n/a	26	36
8	Bouzoubaâ *et al.*	2000, 2001	55-58	370	0.33	SP	30	42
9	Bouzoubaâ and Lachemi	2001	40, 50, 60	400	0.35-0.45	SP	26-48	n/a
10	Yin *et al.*	2002	30-50	580	0.23	SP	95.2	n/a
11	Xie *et al.*	2002	30	540	0.27-0.29	SP	71.1	n/a
12	Jaturapitakkul *et al.*	2004	15, 25, 35, 50	554	0.27	SP	77.3-82.5	86 – 96
13	Siddique	2004	40, 45, 50	400	0.4-0.41	SP	23-26.7	28-34
14	Oner *et al.*	2005	15, 20, 25, 33, 42, 50, 58	320	0.74-0.8	n/a	39.3-42.7	n/a

Table 2.3 Summary of the literature review on fly ash and its effect on the concrete compressive strength (continued)

16	Atiş	2005	50, 70	400	0.28-0.4	SP	19-70	30-83.7
17	Sata *et al.*	2007	10, 20, 30, 40	560	0.28	SP	78-84	86-92
18	Shafiq *et al.*	2007	30, 40	325	0.515-0.560	n/a	25	35

n/a: Not available

2.4.3.1 Effects of Fly Ash on the Modulus of Elasticity

Since the modulus of elasticity of concrete is related to its compressive strength, in general, the effect of fly ash on the elastic modulus of concrete is similar to the effect of fly ash on strength development.

Haque and Kayali, (1998) performed an experiment in which six mixtures were cast with total cementitious material content of 500 kg/m^3. They used fine fly ash; the replacement of fine fly ash was 0%, 10% and 15% by weight of cement, the water/cementitious material ratio varied between 0.25-0.38. The concretes with 10% fine fly ash exhibited higher early strength followed by an excellent development of strength over time. The 28-day compressive strength of 500-10 and 500-15 concretes were 111 and 102 MPa, respectively. The addition of fine fly ash also resulted in an increase in the modulus of elasticity.

Siddique, (2004) used high volume of class F fly ash. Portland cement was replaced with 0%, 40%, 45% and 50% of class F fly ash. The cementitious material content for all the concrete mixes was 400 kg/m^3, the water/cementitious ratio was 0.4-0.41. The results showed that the use of high volumes of class F fly ash as a partial replacement (40%, 45% and 50%) of cement in concrete decreased its 28-day compressive strength, a reduction of 28%, 34% and 38% respectively in comparison with the control mixture was obtained. The concrete compressive strengths at 28 and 91 days were 23-26.7 MPa and 28-34 MPa respectively. Also the 28-day modulus of elasticity was reduced. The results at 91 and 365 days indicated that there was continuous and significant improvement in strength beyond the age of 28 days. The significant increase in strength of high-volume fly ash is due to the pozzolanic reaction of fly ash. And he concluded that 40% and 45%

mixtures could very well be used for structural concrete and 50% mixture could be used for general concrete construction.

2.5 CFRP Composites and Fly Ash

Ramana *et al.*, (2000) used the CFRP composites to strengthen the reinforced concrete beams made of fly ash. 20% fly ash by weight of cement was added to improve the workability and compressive strength that was 30 MPa. Four sets of beams, three with different amounts of CFRP composite reinforcement by changing the width of laminate, and one without CFRP composite were tested in four-point bending over a span of 900 mm. the increase in strength and stiffness was assessed. The first crack and ultimate moments of strengthened beams were significantly higher than that of the control beam indicating the reinforcing effect of the CFRP composite laminates. The maximum increase in the first crack and ultimate moments were about 150 and 230%, respectively. There was a substantial increase in the stiffness of the strengthened beams and the maximum increase was 110%. The flexural strength of beams was significantly increased as the width of the laminate increased.

2.6 Summary of the Literature Review

The objective of this chapter was to determine the possible gaps in the existing research studies dedicated on the application of CFRP plates for flexural strengthening of RC beams made of different types of concrete. Based on the detailed review of literature as presented, following summary was helpful to formulate the objectives of the current study:

- FRP composites have shown superior properties as compared to that of steel plates as well as GFRP and AFRP composites.
- Optimum length of CFRP plate for optimum flexural strengthening has not been fully addressed in the existing literature. Very few studies have been conducted to investigate the optimum length of CFRP plate that could be used for flexural strengthening of reinforced concrete beams with high load carrying capacity; in general it was achieved with full span length of fibre in normal concrete beams.
- Most of the literature shown that high-strength concrete such as 40 MPa onwards was achieved at high cement content of 450-700 kg/m^3.
- Most researchers used the CFRP composites to strengthen the reinforced concrete beams of normal concrete (cement, sand and aggregates) and not of blended cement concrete.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 Introduction

There were two major parts in the methodology of this research. In the first part concrete mixes were established that contained fly ash as a cement replacement material with 20%, 30% and 40% by mass of cement; hence 28-day target strength between 50-60 MPa could be achieved. The second part investigated the flexural capacity of beams externally strengthened with CFRP strips.

3.2 Materials

3.2.1 Cement

Ordinary Portland cement was supplied by YTL Cement Berhad, Malaysia. It conformed to BS EN 197-1. Chemical properties of OPC are shown in Table 3.1.

Table 3.1 Chemical properties of Ordinary Portland Cement

Chemical constituents	Value (%)
Silicon dioxide (SiO_2)	21.98
Aluminum dioxide (Al_2O_3)	4.65
Ferric oxide (Fe_2O_3)	2.27
Calcium oxide (CaO)	61.55
Magnesium oxide (MgO)	4.27
Sulfur tri-oxide (SO_3)	2.19
Potassium dioxide (K_2O)	1.04
Sodium dioxide (Na_2O)	0.11

3.2.2 Fly Ash (FA)

The fly ash used in this study was obtained from Manjung Power Station at Lumut, Perak, that was classified as low lime fly ash or ASTM Class F fly ash. Chemical composition and physical properties of fly ash are given in Table 3.2.

Table 3.2 Chemical composition and physical properties of fly ash

Property	Value (%)
Chemical constituents	
Silicon dioxide (SiO_2)	56.39
Aluminum dioxide (Al_2O_3)	17.57
Ferric oxide (Fe_2O_3)	9.07
Calcium oxide (CaO)	11.47
Magnesium oxide (MgO)	0.98
Sulfur tri-oxide (SO_3)	0.55
Sodium dioxide (Na_2O)	1.91
Potassium dioxide (K_2O)	1.98
Loss on ignition	1.85
Physical properties	
Specific gravity	2.37
Fineness, (m^2/kg)	243

3.2.3 Sand

Mining sand from Tronoh, Malaysia, was used as fine aggregate for all mixes. The sieve analysis test for fine aggregate is shown in Table 3.3. The particle size distribution curve for fine aggregate is shown in Figure A1 in the appendices.

Table 3.3 Sieve analysis of sand

Sieve Size	Weight Retained (g)	% Retained	% Cumulative Weight Retained	% Toal Passing
2.36 mm	110.3	11.03	11.03	88.97
2 mm	30.4	3.04	14.07	85.93
1.18 mm	118.1	11.81	25.88	74.12
0.6 mm	230.2	23.02	48.9	51.1
0.425 mm	125.2	12.52	61.42	38.58
0.3 mm	135.1	13.51	74.93	25.07
0.15 mm	180.3	18.03	92.96	7.04
0.075 mm	60.2	6.02	98.98	1.02
Pan	10.2	1.02	100	0

Total = 1000 g

3.2.4 Coarse Aggregate

Granite gravel with a maximum particle size of 20 mm that was obtained from Papan granite, Ipoh, Malaysia was employed. The sieve analysis for coarse aggregate is shown in Table 3.4. Both fine and coarse aggregates conformed to BS 882: 1992. The particle size distribution curve for coarse aggregate is shown in Figure A2 in the appendices.

Table 3.4 Sieve analysis of coarse aggregates

Sieve Size	Weight Retained (g)	% Retained	% Cumulative Weight Retained	% Toal Passing
20 mm	77	2.82	2.82	97.18
14 mm	579	21.17	23.99	76.01
10 mm	780	28.52	52.50	47.50
5 mm	1052	38.46	90.97	9.03
3.35 mm	102	3.73	94.70	5.30
Pan	145	5.30	100.00	0.00

Total = 2735 g

3.2.5 Superplasticizer

The superplasticizer used was categorized as water-reducing agent in that they are formulated from materials that allow much greater water reductions. Commercially available, Sikament-NI, naphthalene formaldehyde sulphonate superplasticizer in the form of aqueous solution was used as water reducing admixture (WRA) for all concrete mixes according to BS 5075: Part 3: 1985.

3.2.6 Reinforcing Steel

The longitudinal reinforcement used was high yield steel deformed bars of 16 mm diameter that possessed the yield strength, modulus of elasticity, ultimate strain of 460 MPa, 230 GPa and 0.002 respectively. The shear reinforcement consisted of vertical stirrups of 6 mm diameter mild steel bars having characteristics strength of 250 MPa.

3.3 Development of High Strength Concrete Mix Design Using Fly Ash

For the purpose of establishing the concrete mix with low cementitious material and to achieve compressive strength of at least 50 MPa at 28 days with acceptable slump, trial mixes were prepared in compliance with the required slump (80-150) mm and targeted strength (50 MPa). Table 3.5 shows the values of measured slump, w/c ratio, cementitious material content and the superplasticizer content for various trial mixes.

Optimum cement content was selected with the aim of cost reduction; therefore trial contents were 335 kg/m^3, 360 kg/m^3 and 380 kg/m^3. By the partial replacement of cement the net cement content was further reduced. The mix proportions for different cement content are shown in Table 3.6

Table 3.5 Trial mixes and measured slump

Mix proportions (OPC: Sand: Aggregate)	w/b	SP (% of binder content)	Slump (mm)
1:2:3.2	0.32	1.0	0
1:2:3.2	0.32	1.5	0
1:2:3.2	0.32	2.0	0
1:2:3.2	0.36	1.0	0
1:2:3.2	0.36	1.5	0
1:2:3.2	0.36	2.0	12
1:2:3.2	0.4	1.0	20
1:2:3.2	0.4	1.5	40
1:2:3.2	0.4	2.0	95
1:2.25:3.5	0.45	0	10
1:2.25:3.5	0.45	1.0	20
1:2.25:3.5	0.45	1.5	45
1:2.25:3.5	0.45	2.0	65

Table 3.6 Concrete mix proportions

Mix ID	w/b	SP (% of binder content)	Slump (mm)	Water (kg/m^3)	OPC (kg/m^3)	A (kg/m^3)	Sand (kg/m^3)	Aggregate (kg/m^3)
M 335/0	0.45	2.0	65	151	335	0	753	1172
M 335/20	0.45	2.0	70	151	268	67	753	1172
M 360/0	0.4	2.0	95	144	360	0	720	1152
M 360/20	0.4	2.0	105	144	288	72	720	1152
M 360/30	0.4	2.0	122	144	252	108	720	1152
M 360/40	0.4	2.0	155	144	216	144	720	1152
M 380/0	0.4	2.0	140	152	380	0	760	1216
M 380/20	0.4	2.0	120	152	304	76	760	1216
M 380/30	0.4	2.0	142	152	266	114	760	1216
M 380/40	0.4	2.0	160	152	228	152	760	1216

Where:

OPC: ordinary Portland cement

FA: fly ash

w/b: water/binder ratio

SP: superplasticizer

3.3.1 Mixing, Casting and Curing of Concrete

All concrete ingredients were mixed as specified in BS 8110: 1997. To determine the workability of the fresh mixed concrete, slump test was performed according to BS 1881: Part 102: 1983. After achieving the required workability of concrete, the specimens were cast to determine the hardened concrete properties.

150-mm concrete cubes were cast for compressive strength, 100 mm diameter × 200 height cylinders were cast for modulus of elasticity and 150 mm wide, 200 mm deep, and 2000 mm length reinforced concrete beams were cast for flexural test. After casting, plastic sheets were used to cover the specimens to prevent the water from evaporating. After 24 hours, the specimens were demoulded and placed in water for curing process according to BS 1881: Part 108: 1983.

3.3.2 Hardened Concrete Tests

The hardened concrete tests were performed for two purposes: compressive strength and modulus of elasticity.

3.3.2.1 Compressive Strength

The compression test was performed on 150-mm cubes and the compressive strength was determined on three cubes at 3, 7, 28, and 90 days curing. In the compression test, the cube, while still wet, was placed with the cast faces away from the platens of the testing machine; the load on the cube was applied at a constant rate of 6.8 KN/s according to BS 1881: Part 116: 1983.

3.3.2.2 Modulus of Elasticity

Compressive strength was determined on three cylinders for every mix at 28 and 90 days curing. In the compression test the top surface of the cylinder was in contact with the platen of the testing machine; the load was applied at a constant rate of 2.4 KN/s according to BS 1881: Part 110: 1983.

The calculation of the modulus of elasticity was based on ACI 318-95 by using the equation below:

$$E_c = 4.73\sqrt{fc'} \qquad 3.1$$

Where:

E_c: Modulus of elasticity (GPa).

fc' : Cylinder compressive strength (MPa).

3.4 Strengthened Beams

In total, eight reinforced concrete beams were fabricated; six of them were strengthened for flexure using CFRP strips. The other two were tested without CFRP and designated as the control beams. This section describes the philosophy of the design, the CFRP application and instrumentation.

3.4.1 Carbon Fibre Reinforced Polymer

A composite consists of two or more materials that are combined to produce a product of enhanced individual properties. In particular, FRP is a combination of high-strength fibres and a matrix. The fibres represent the strength of the composite, and the matrix is the product that holds the fibres together and acts as a load transfer median. The carbon fibres are stronger and stiffer than most of the active fibres, more corrosion resistant, lower in density and more widely available as a raw material.

3.4.1.1 CFRP Strips

The carbon fibre reinforced polymer (CFRP) strips used in this research were supplied by Sika, Malaysia and known as CarboDur S1012. The size was 100 mm wide and 1.2 mm thick. Important properties of Sika CFRP strips are appended in Table 3.7

Table 3.7 Properties of Sika's CFRP system

	Tensile strength (MPa)	Elongation at break (%)	Elastic modulus (GPa)	Compressive strength (MPa)	Adhesive strength on concrete (MPa)	Adhesive strength on steel (MPa)
Sika CFRP strip [a]	>2400	1.4	150	_	_	_
SikaDur – 30 adhesive	_	_	12.8	> 100	> 2	> 25

3.4.1.2 Bonding Material

The strips were bonded to the soffit of the beams using epoxy material supplied by Sika, Malaysia, known as SikaDur-30. The epoxy consisted of two components; part A was a white coloured base; part B was a dark grey coloured hardener. The mix ratio of part A and part B is three to one by weight. The mixture of the two is light grey in colour. The properties of the Sikadur-30 adhesive are shown in Table 3.7.

3.4.2 Design of Beams

The beams were designed to study their flexural behaviour with different level of strengthening. All beams were 150 mm in width, 200 mm in height and 2150 mm in length to achieve a clear span of 2000 mm. The concrete cover for steel was 25 mm giving an effective depth of 161 mm. Two 16 mm diameter bars were used as tensile reinforcement. A critical part of the design was ensuring the beams failed under flexure but not in shear. To accomplish this, the shear stirrups were closely spaced; stirrups of 6 mm diameter were used for shear strengthening and spaced at 100 mm centre to centre.

The design was based on BS 8110: Part 1: 1997. Beam dimensions with longitudinal reinforcement and stirrup spacing are shown in Figure 3.1. The specimens were cast in a plywood formwork. Prior to casting, the walls of the formwork were lubricated with oil to prevent adhesion with the cured concrete. The concrete was vibrated and kept in a moist environment by using plastic sheets. Specimens were demoulded after twenty four hours, and sprayed with water every day.

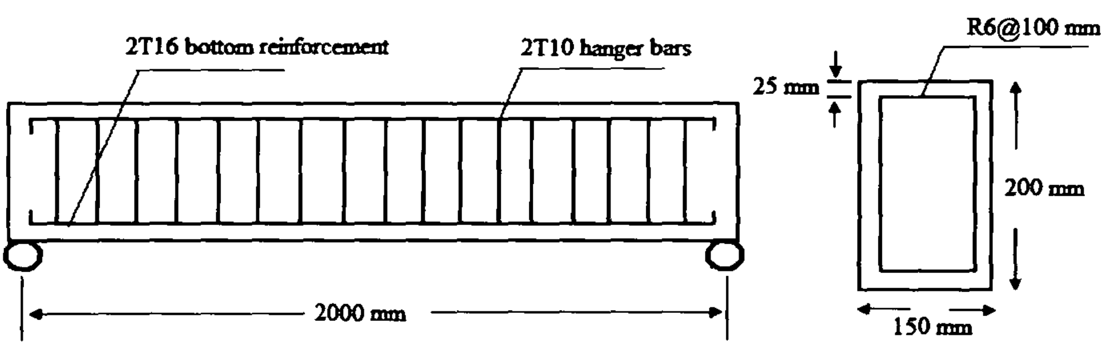

Figure 3.1 Beam dimensions and reinforcement details

3.4.2.1 Installation of CFRP Strengthening System

The most important factor in creating a composite system with reinforced concrete beam and CFRP is assuring the bond between the materials is adequate. Preparation of the concrete surface and the application of the CFRP are discussed in the following sections.

3.4.2.1.1 Concrete Surface Preparation

It was necessary to have a level concrete surface to serve as a bonding plain for the CFRP. Also, the surface should be independent from all unwanted particles such as dust or grease. To achieve these, an electrical and nonelectrical hand held steel grinder was used to remove the weak surface layer on the tension side of the beam. The preparation was complete by blowing the specimen with compressed water to remove any excess particles.

3.4.2.1.2 Bonding of the CFRP

To investigate the effective length of CFRP plate for optimum strengthening, three different lengths were chosen. For each studied length two beams were constructed. The CFRP plates were pasted to the soffit of the beams after curing period of at least 28 days according to the supplier's recommendations. The procedure of bonding the CFRP to the beams is discussed below:

- The CFRP plates were cut to the required length.
- The CFRP plates were cleaned with acetone, this process was repeated until the washcloth was no longer blackened.
- The reinforced concrete beams were first inverted, so that the tension face was at the top, to simplify the application.
- The epoxy was hand-mixed thoroughly and applied evenly to both carbon fibre strip and the concrete surface using a roller brush.
- The CFRP strip was then smoothly hand-laid to achieve wrinkle-free surface, and the extra epoxy was squeezed out and removed keeping the thickness of epoxy between 2-3 mm.
- After installation of the CFRP plates, the specimens were cured at room temperature for at least two weeks before testing.

3.4.2.2 Details of the Strengthened Beams

Beams CB1 and CF1 were served as control beams without any CFRP plate strengthening. Beam CB was made of normal concrete with a compressive strength of 66 MPa. The mix proportions for M 380/0 were chosen to fabricate all normal concrete beams to provide a concrete compressive strength of 66 MPa. Beams in CF1-series were made according to the mix proportions of M 380/30 that incorporated 30% fly ash by weight. The compressive strength for this mix was 69 MPa. The 30% fly ash was chosen to examine the behaviour of reinforced concrete beams at higher volume of fly ash.

Beam CB2 was normal concrete strengthened with a CFRP plate length of 1333 mm (67% of the span length), this length was chosen because it represents the region of the pure bending. Beam CF2 was made of fly ash concrete strengthened with CFRP plate of 1333 mm, same as beam CB2. The external reinforcement of these two beams is shown in Figure 3.2.

Beam CB3 was made of normal concrete and strengthened with a CFRP plate of length of 1667 mm (83% of the span length) which is the average length of 1333 mm and the full span length, (2000 mm). Beam CF3 was made of fly ash concrete and strengthened with a CFRP plate of length 1667 mm as well. The external reinforcement of CB3 and CF3 is shown in Figure 3.3.

Beam CB4 was made of normal concrete; it was strengthened with a CFRP plate of length 2000 mm corresponding to the full span length. Beam CF4 was made of fly ash concrete and it was strengthened with a CFRP plate of a length of 2000 mm as well. The external reinforcement of CB4 and CF4 is shown in Figure 3.4. The summary of all the beams is shown in Table 3.8.

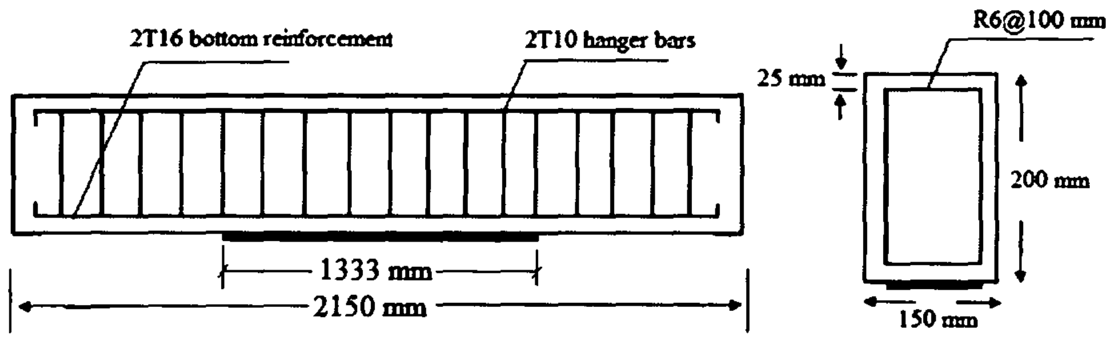

Figure 3.2 External reinforcement for CB2 and CF2

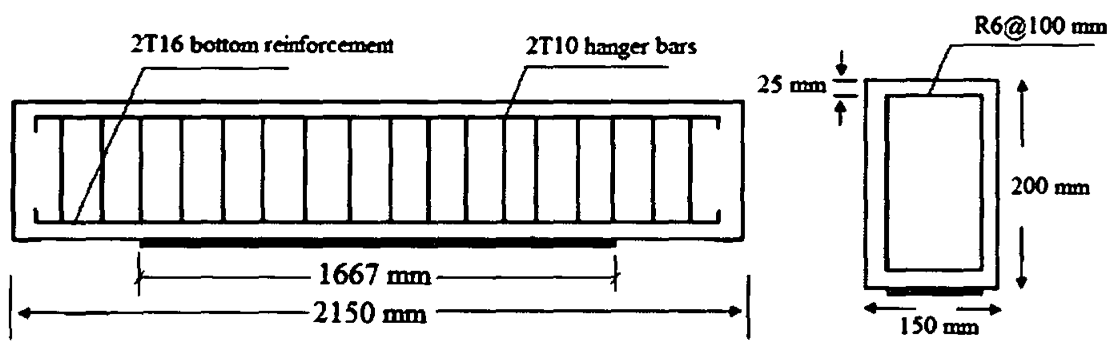

Figure 3.3 External reinforcement for CB3 and CF3

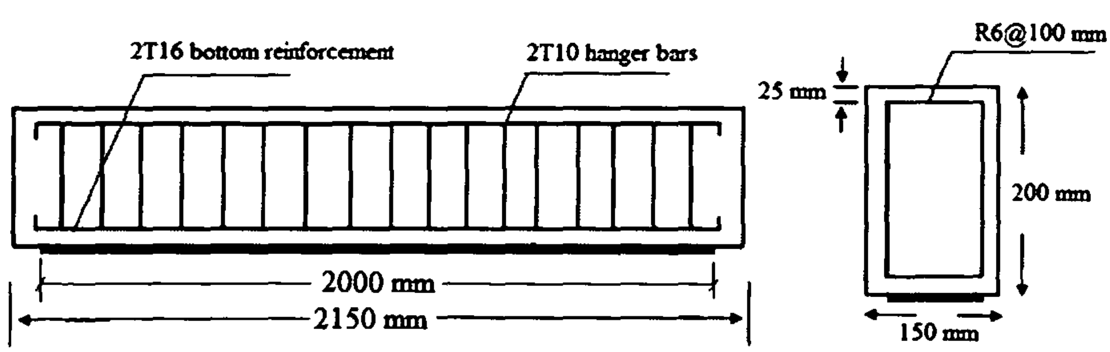

Figure 3.4 External reinforcement of CB4 and CF4

3.4.3 Test Set-up and Instrumentation

The deflection of the constructed beams was measured using a Linear Variable Displacement Transducer (LVDT). The deflection readings were recorded every 5 seconds. Figure 3.5 shows the LVDT position. A mid-span point load was exerted on all eight test beams. The test set-up is illustrated in Figure 3.5. The beams were subjected to a static point loading using an MTS hydraulic actuator (500 KN capacity) at the mid-span of the beam, the rate of loading was 0.2 KN/s. Table 3.8 list the tested beams details including test parameters for each of the eight specimens. These include; the specimen ID, fly ash content, area of CFRP and the length of CFRP plate, properties of steel, concrete and CFRP.

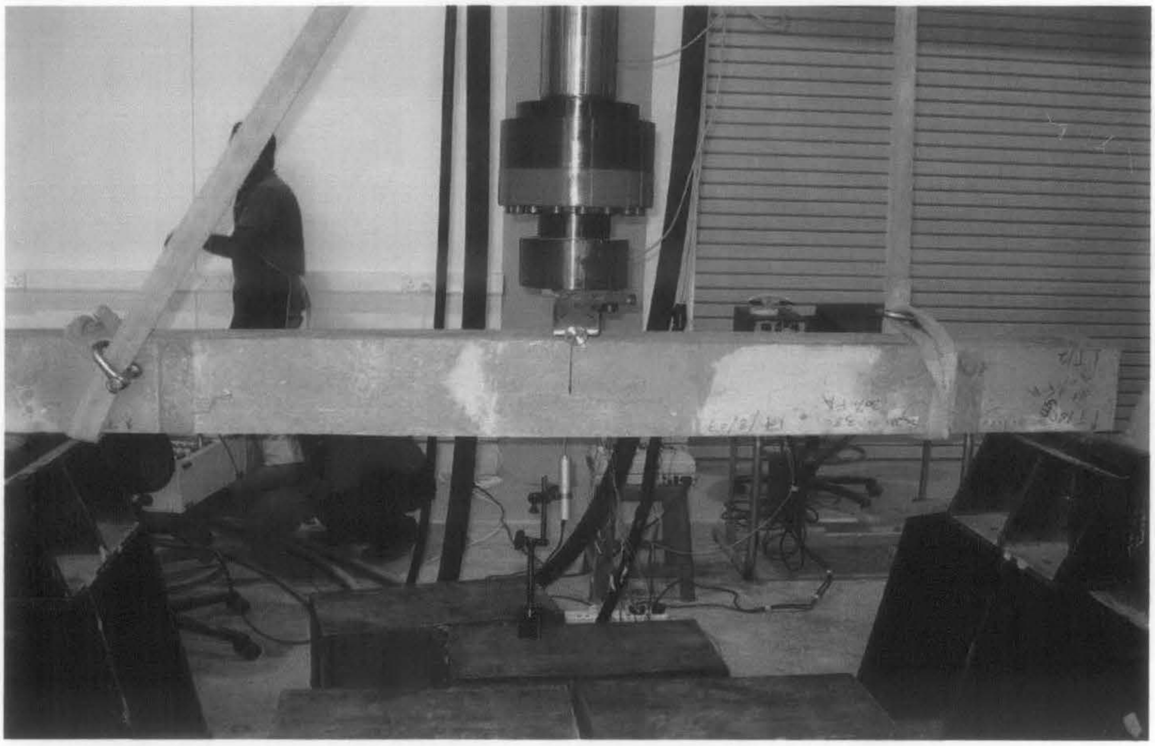

Figure 3.5 Test set-up

Table 3.8 Details of the tested beams

Beam ID	Beam Dimensions					Concrete Properties			Tension Reinforcement			FRP Properties				
	b	h	d	L	a	FA	f_{cu}	E_c	f_y	E_s	A_s	L_{frp}	L_{frp}	A_f	E_f	ε_{fu}
	mm	mm	mm	mm	mm	(%)	(MPa)	(GPa)	(MPa)	(GPa)	mm^2	mm	(%)	mm^2	(GPa)	(%)
CB1	150	200	161	2150	1000	0	66	34	460	230	402	0	0	120	150	1.4
CF1	150	200	161	2150	1000	30	69	34	460	230	402	0	0	120	150	1.4
CB2	150	200	161	2150	1000	0	66	34	460	230	402	1333	67	120	150	1.4
CF2	150	200	161	2150	1000	30	69	34	460	230	402	1333	67	120	150	1.4
CB3	150	200	161	2150	1000	0	66	34	460	230	402	1667	83	120	150	1.4
CF3	150	200	161	2150	1000	30	69	34	460	230	402	1667	83	120	150	1.4
CB4	150	200	161	2150	1000	0	66	34	460	230	402	2000	100	120	150	1.4
CF4	150	200	161	2150	1000	30	69	34	460	230	402	2000	100	120	150	1.4

3.5 Analysis Equations

3.5.1 Equations from published data

The analysis of all the eight beams was based on Toutanji *et al.*, (2006) according to Figure 3.6.

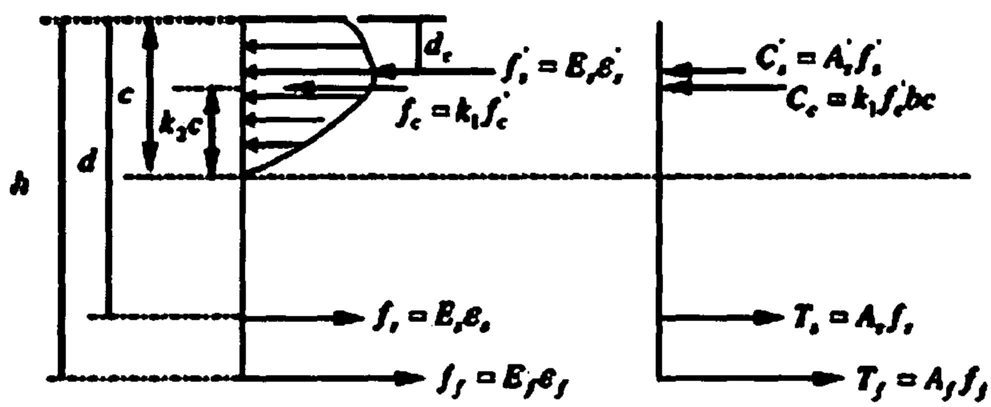

Figure 3.6 Stresses and forces of beam cross-section

All the following equations are based on Toutanji et al., (2006):

$$C_c = T_s \text{ ------------------------------------ } 3.2$$

$$C_c = T_s + T_f \text{ -------------------------------- } 3.3$$

$$k_1 = 500\varepsilon_c(1 - 166.7\varepsilon_c) \text{ -------------------- } 3.4$$

$$k_2 = \frac{2 - 375\varepsilon_c}{3 - 500\varepsilon_c} \text{ ------------------------------ } 3.5$$

$$\varepsilon_f = \frac{h - c}{d - c}\varepsilon_y \text{ -------------------------------- } 3.6$$

$$\varepsilon_c = \frac{c}{d-c}\varepsilon_y \leq 0.003 \text{ ------------------------ } 3.7$$

$$\varepsilon_y = \frac{f_y}{E_s} \text{ -- } 3.8$$

c is determined by:

$$Ac^2 + Bc + C = 0 \text{ ------------------------------ } 3.9$$

where

$A = k_1 f_c' b$

$B = \varepsilon_c (A_s' E_s + A_f E_f) - A_s f_y$

$C = -\varepsilon_c \left[A_s' E_s d_c + A_f E_f h\right]$

$$M_u = k_1 k_2 f_c' bc^2 + A_s' E_s \varepsilon_s' (c - d_c) + A_s f_y (d - c) + A_f E_{ff} (h - c) \text{ ------------ } 3.10$$

$$M_y = k_1 k_2 f_c' bc^2 + A_s' E_s \varepsilon_s' (c - d_c) + A_s f_y (d - c) + A_f E_f \varepsilon_f (h - c) \text{ ---------- } 3.11$$

$$\varepsilon_{ff} = k_m \varepsilon_{fu} \text{ -- } 3.12$$

$$\frac{1}{60\varepsilon_{fu}}\left(1 - \frac{nE_f t_f}{360000}\right) \leq 0.9 \rightarrow nE_f t_f \leq 180000 \text{ ------------------------------- } 3.13$$

$$\Delta_u = \frac{M_u}{24 E_c I_{cr}}(3L^2 - 4a^2) \text{ -- } 3.14$$

$$\Delta_y = \frac{M_y}{24 E_c I_{cr}}(3L^2 - 4a^2) \text{ -- } 3.15$$

$$I_{cr} = \frac{1}{3}bc^3 + \frac{E_f}{E_c}A_f(h-c)^2 + \frac{E_s}{E_c}A_s(d-c)^2 + \frac{E_s}{E_c}A_s'(c-d_c)^2 \text{------------------ } 3.16$$

$$\phi = \frac{\varepsilon_c}{c} \text{ --}3.17 \text{ [Chahrour and Soudki, (2005)].}$$

$$a = KL^2\phi \text{ ---}3.18 \text{ [Mosley et al., (1996)].}$$

3.5.2 Equations proposed by ACI 440.2R-02

$$\varepsilon_{fe} \le k_m \varepsilon_{fu} \text{ --}3.19$$

$$k_m = \frac{1}{60\varepsilon_{fu}}\left(1 - \frac{nE_f t_f}{360000}\right) \text{ ---}3.20$$

$$M_n = A_s\delta_s\left(d - \frac{\gamma c}{2}\right) + \Psi_{frp}A_f E_f \varepsilon_{fe}\left(d_f - \frac{\gamma c}{2}\right) + A'_s\delta'_s\left(\frac{\gamma c}{2} - d'\right)\text{-------------}3.21$$

$$M_u = \varphi M_n \text{---}3.22$$

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents and discusses the results of the experimental program. In section 4.2, the results of work on development of concrete mixes with target strength 50-60 MPa using FA as partial replacement is presented and discussed. The effect of different binder contents and age on the compressive strength, the variation of modulus of elasticity for different binder contents at 28 and 90 days are presented. In section 4.3, the flexural behaviour of RC beams strengthened with CFRP strips and the control beams are presented and discussed. The theoretical ultimate loads and elastic deflections are compared in section 4.3.4 with experimental values.

4.2 Development of Concrete Mix for 28 days Target Strength of 50-60 MPa

In order to determine the optimum fly ash content three replacement levels were tried and the compressive strength results at different ages were compared with the control mix having 0% fly ash.

For the cementitious material content of 335 kg/m^3, when the cement was replaced with 0% and 20% fly ash the 28-day compressive strength was 39.5 MPa and 46.2 MPa respectively which were lower than 50 MPa and the mix was rejected (did not fulfill the objectives). The development of the strength and the calculated moduli of elasticity for all mixes at different ages are shown in Table 4.1.

Table 4.1 Strength development and modulus elasticity of the concrete mixes at different ages

Mix ID	Compressive Strength (MPa)				Modulus of Elasticity (GPa)	
	3-days	7-days	28-days	90-days	28-days	90-days
M 360/0	28.5	40.6	50.1	61.8	26.5	30.3
M 360/20	28.4	38.9	61.4	74.9	30.1	35.3
M 360/30	26.5	31.5	51.0	67.3	29.0	30.8
M 360/40	21.4	24.7	37.8	55.0	23.9	30.0
M 380/0	42.8	47.2	60.0	66.1	29.0	33.6
M 380/20	38.4	43.1	68.4	84.2	33.2	36.6
M 380/30	34.2	39.4	56.4	69.0	30.0	34.0
M 380/40	29.3	36.9	52.1	69.6	26.1	31.5

4.2.1 Effects of Fly Ash with Binder Content of 360 kg/m^3

The control mix was designated M 360/0 (1/2), where the variable 1 represents the binder content and variable 2 represents the fly ash percentage. The mix proportions were 1:2:3.2:0.4, for the control sample without fly ash replacement. The 28 and 90-day compressive strengths for this mix were 50.1 MPa and 61.8 MPa respectively. The effects of fly ash content on the compressive strength for this binder content are shown in Figure 4.1.

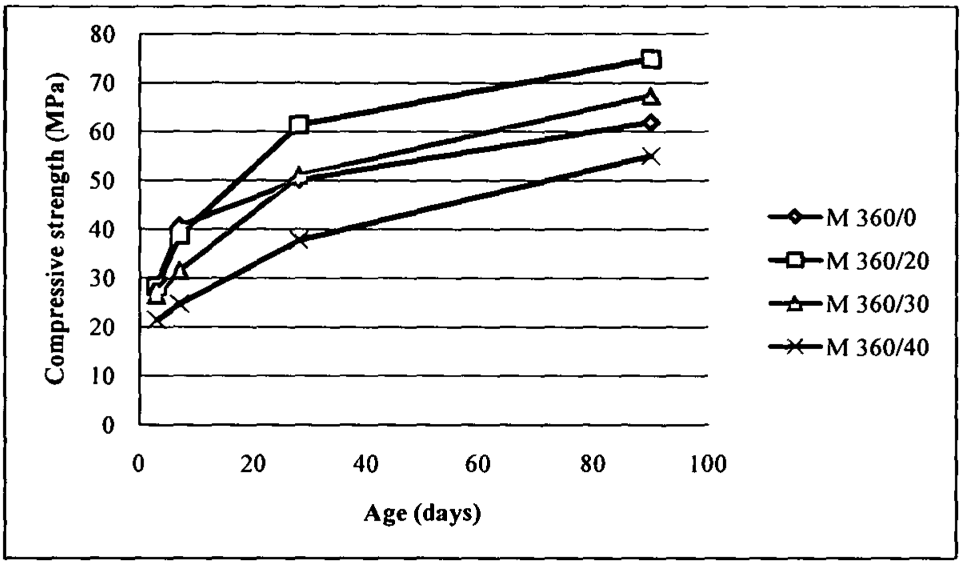

Figure 4.1 Effects of fly ash content on the compressive strength of concrete with a binder content of 360 kg/m^3 at different ages

When the cement was replaced with 20% fly ash the compressive strength of fly ash specimens were similar to or slightly lower than the control at 3 and 7 days, but were higher at 28 days and 90 days. The 28 and 90-day compressive strength for 20% fly ash concrete were 61.4 MPa and 74.9 MPa respectively, which represent 22.6% and 21.1% higher than that of the control mix. 20% fly ash replacement showed the highest compressive strengths at all ages over the other fly ash mixes for 360 kg/m^3 binder content.

When the cement was replaced with 30% fly ash, the compressive strength of the fly ash mixes was lower than the control mix at the ages of 3 and 7 days but similar and higher compressive strength was obtained at 28 and 90 days respectively; the reduction in the compressive strength at early ages is attributed to the slow pozzolanic reactions of fly ash and lower cement content. The 28 and 90-day compressive strengths for this mix were 51

MPa and 67.3 MPa respectively. The 28-day compressive strength for this mix was similar to that of the control mix. The 90-day compressive strength was 9% higher than the control specimen.

The mix with 40% fly ash replacement exhibited lower compressive strengths at all ages when compared to the control mix. The 28-day compressive strength for this mix was 37.8 MPa, which was 24.6 % lower than the control mix. Although at 28 days, the replacement of cement with 40% fly ash decreased the compressive strength of concrete, but, even then, this compressive strength could very well be used for structural concrete. The 90-day compressive strength was 55 MPa which was 11% lower than that of the control. The negative effect of using fly ash on concrete strength appeared to be insignificant because 55 MPa still can be considered as high-strength concrete.

4.2.2 Effects of Fly Ash with Binder Content of 380 kg/m^3

The control mix was designated M 380/0 and the mix proportions for this binder content were 1:2:3.2:0.4. The control mix in which the fly ash replacement was 0% yielded 28 and 90-day compressive strengths of 60 MPa and 66.1 MPa respectively. The effects of fly ash content on the compressive strength for 380 kg/m^3 binder content are shown in Figure 4.2.

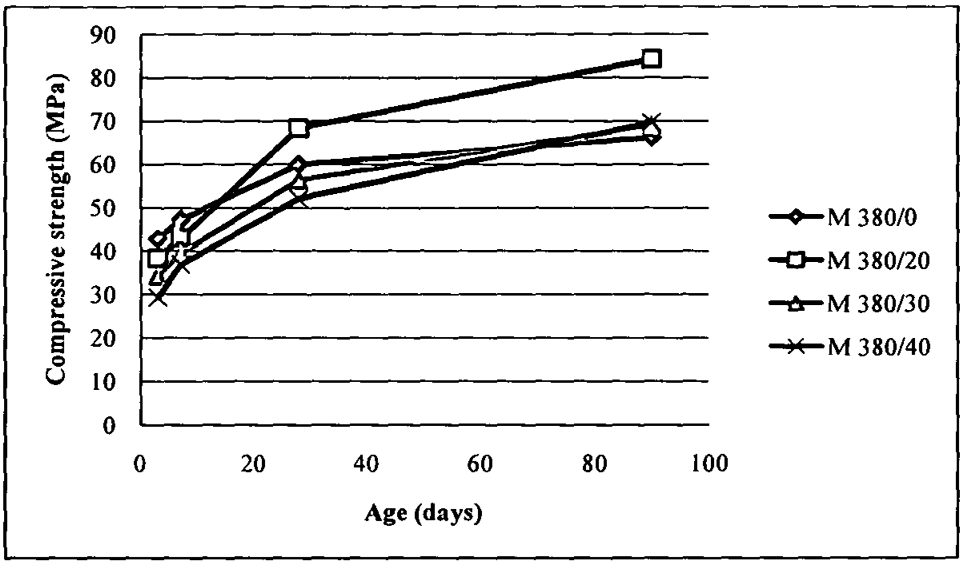

Figure 4.2 Effects of fly ash content on the compressive strength of concrete with a binder content of 380 kg/m^3 at different ages

When 20% of fly ash replaced the cement, the compressive strength at the ages of 3 and 7 days was slightly lower than the control mix. The 28 and 90-days compressive strengths were 68.4 MPa and 84.2 MPa respectively which were 14% and 40% higher than that of the control mix. It is worth mentioning that 20% fly ash replacement gained the highest compressive strength over all the concrete mixes which indicated that the optimum replacement of cement by fly ash is 20%.

The 3 and 7-day compressive strength for 30% fly ash content was 34.2 MPa and 39.4 MPa respectively which was 20% and 16.4% lower than that of the control mix. The 28-day strength of 56.4 MPa and 6% lower than the control mix. The 90-day compressive strength was 69 MPa which was 4% higher than the control, indicated that the compressive strength for the 30% fly ash content-mix and the control mix are almost similar at the ages of 28 and 90 days.

The 3, 7 and 28-day compressive strengths when the cement was replaced with 40% fly ash were 29.3 MPa, 36.9 MPa, and 52.1 MPa respectively which were 31.5%, 21.8% and 13% lower than that of the control mix. The 90-day compressive strength was 69.6 MPa which was 5.3% higher than the control mix. For 40% fly ash replacement the increase in the strength from 28 to 90 days was about 33.6%, this is due to the pozzolanic characteristics of fly ash.

4.2.3 Effects of Binder Content and w/b ratio

It is worth mentioning that by increasing the binder content from 360 kg/m^3 to 380 kg/m^3, the compressive strength can be increased at all ages, and consequently increases the enhancement that can be produced by using fly ash as a cement replacement material. Figure 4.3 shows the comparison between M 360/0 and M 380/0 at different ages. Also by keeping the water/binder ratio at 0.4, higher compressive strengths can be obtained.

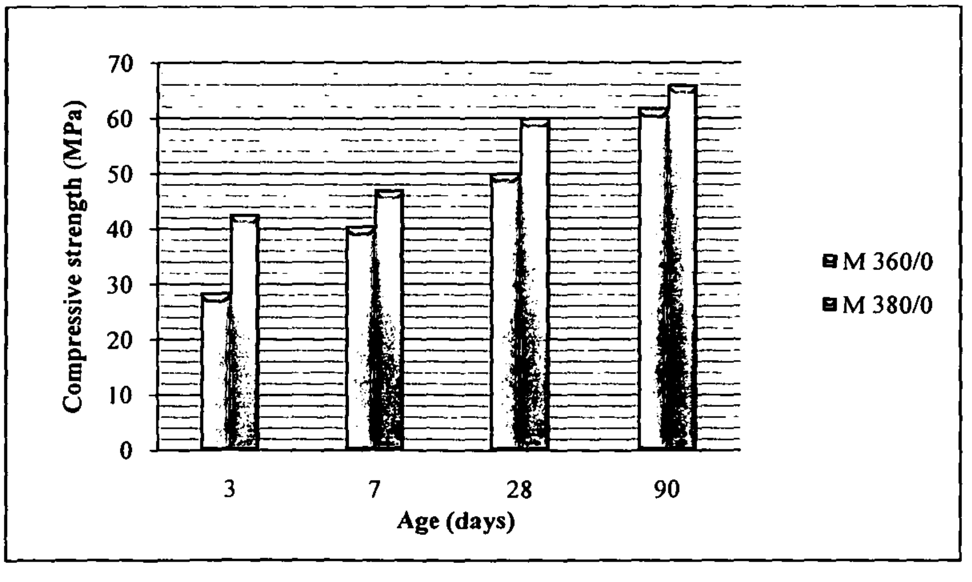

Figure 4.3 Strength development for M 360/0 and M 380/0

4.2.4 Modulus of Elasticity

Modulus of elasticity, E_c of all concrete mixes was calculated using equation 3.1 from the results of cylinder compressive strength measured at 28 and 90 days. Calculated moduli of elasticity, E_c of all concrete mixes are plotted in Figure 4.4.

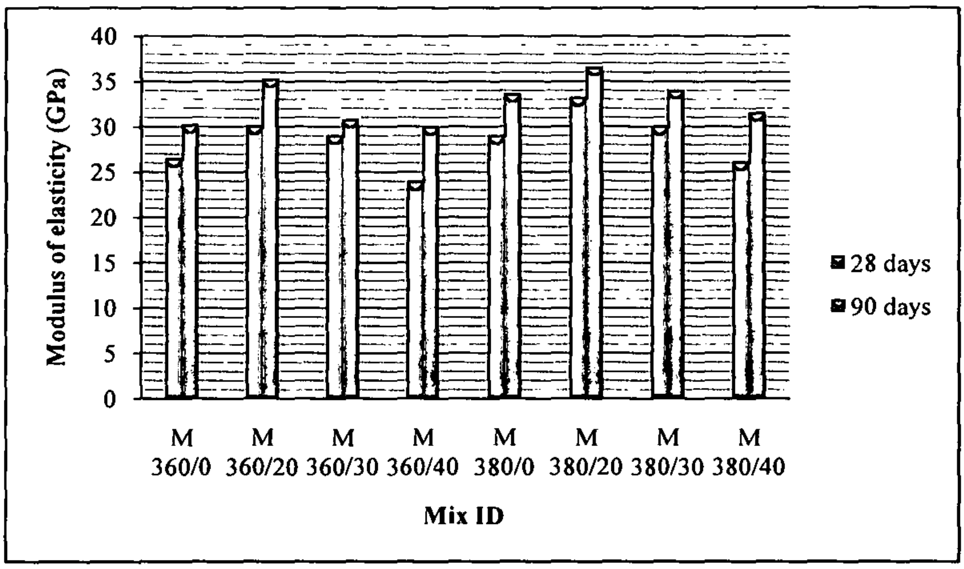

Figure 4.4 Results of the calculated moduli of elasticity

With 20% and 30% fly ash content at the age of 28 days, the modulus of elasticity, E_c obtained was between 29 and 33 GPa which is an indication of better stiffness of concrete. Modulus of elasticity of 20% and 30% fly ash content at the age of 28 days was observed 13.8-24% higher than that of the corresponding control mixes. At the age of 90 days, an average increment of 7% was reported as compared to that measured at 28 days.

4.3 Flexural Behaviour of High Strength Concrete Beams Strengthened with CFRP Strips

In the second part of this study, the flexural behaviour of high strength concrete beams strengthened with CFRP strips was investigated. The principal variable was the length of CFRP strip pasted to the soffit of the beam. The beams were subjected to central point loads. The loads were incremented until failure occurred. The behaviour of the beams was noted. The deflection at each load increment, the crack patterns, and the elastic and ultimate were recorded. Subsequently the coefficient of ductility was calculated. The results are discussed in the following sections. The results in terms of ultimate load, ultimate deflection, service load, and ductility index for the tested beams are presented in Table 4.2.

Table 4.2 Results of ultimate load, ultimate deflection, service load and ductility index for the tested beams

Beam ID	Ultimate load (KN)			Ultimate deflection (mm)		Service load	Ductility index
	P_{test}	$P_{Toutanji}$	P_{ACI}	Δ_{utest}	$\Delta_{uToutanji}$	(KN)	(μ_D)
CB1	54.07	51.4	52.8	25.76	23.20	19.0	4.6
CF1	55.56	51.4	53.0	27.90	24.30	19.4	5.0
CB2	83.03	107.0	73.8	17.68	10.00	27.0	3.5
CF2	83.40	107.7	74.0	16.38	9.96	28.8	3.3
CB3	103.12	107.0	73.8	15.66	10.00	36.0	3.1
CF3	103.89	107.7	74.0	14.82	9.96	38.9	3.0
CB4	106.06	107.0	73.8	13.775	10.00	38.0	2.7
CF4	106.99	107.7	74.0	13.755	9.96	40.9	2.8

4.3.1 Load-Deflection Behaviour

Table 4.3 summarizes the loads carried by the tested beams at service and ultimate levels. The service load was taken at a level at which the deflection of the control beams was measured at about 35% of their ultimate loads, according to Almusallam and AlSalloum, (2001). This means that the service loads for the control, CB1 and CF1 were about 19 KN and 19.4 KN respectively, and the corresponding deflections at that level were about 3.06 mm and 2.91 respectively. Therefore, the values for service load for CB-series and CF-series were taken at reference deflection of 3.06 mm and 2.91 mm respectively.

All the strengthened beams experienced mid-span deflections smaller than those of the control specimens at their failure loads. The values of the maximum deflections decreased as the stiffness of the beam increased due to the increase in the amount of strengthening material. From Figures 4.5 and 4.6, one can observe that CF-series behaved similar to CB-series which means that inclusion of 30% fly ash has no significant effect on the ultimate loads and deflections of the CFRP-strengthened beams. Before the flexural cracks start, the curves are close to each other. After yielding of reinforcement bars, the strength and stiffness of the strengthened specimens were larger when compared to the control specimens. After the failure, the load-deflection curve of the strengthened beams dropped down; this behaviour was expected due to the increase in the beam stiffness as a result of increasing the length of CFRP plates. The ultimate deflections for the tested beams are shown in Figure 4.7.

Table 4.3 Experimental results at service and ultimate levels

Beam ID	Service load (KN)	% gain over the control	Ultimate load (KN)	% gain over the control
CB1	19.0	--	54.07	--
CF1	19.4	--	55.56	--
CB2	27	42	83.03	53.6
CF2	28.76	48	83.40	50
CB3	36.0	89.5	103.12	90.7
CF3	38.9	100.5	103.89	87
CB4	38.0	100	106.06	96
CF4	40.95	111	106.99	92.6

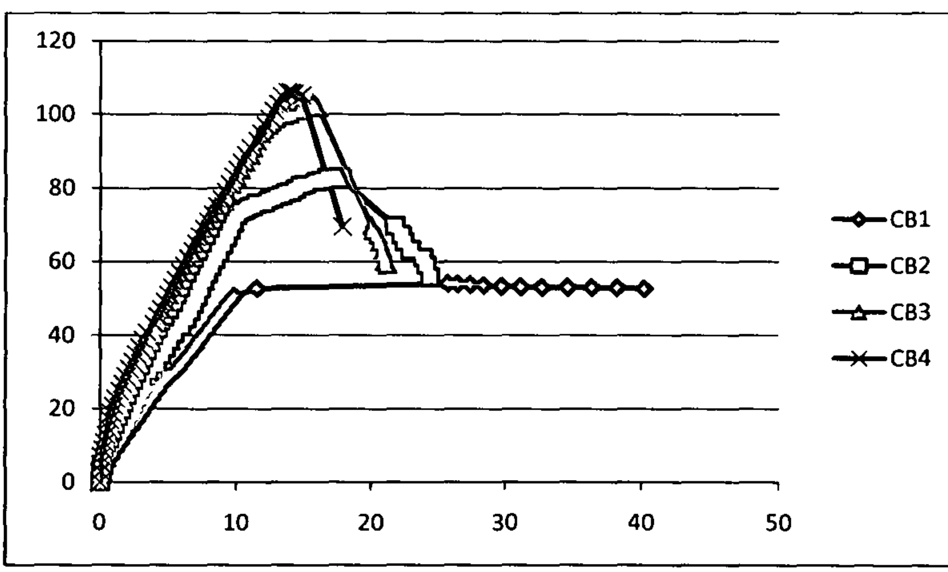

Figure 4.5 Load-deflection relationship for the CB-series

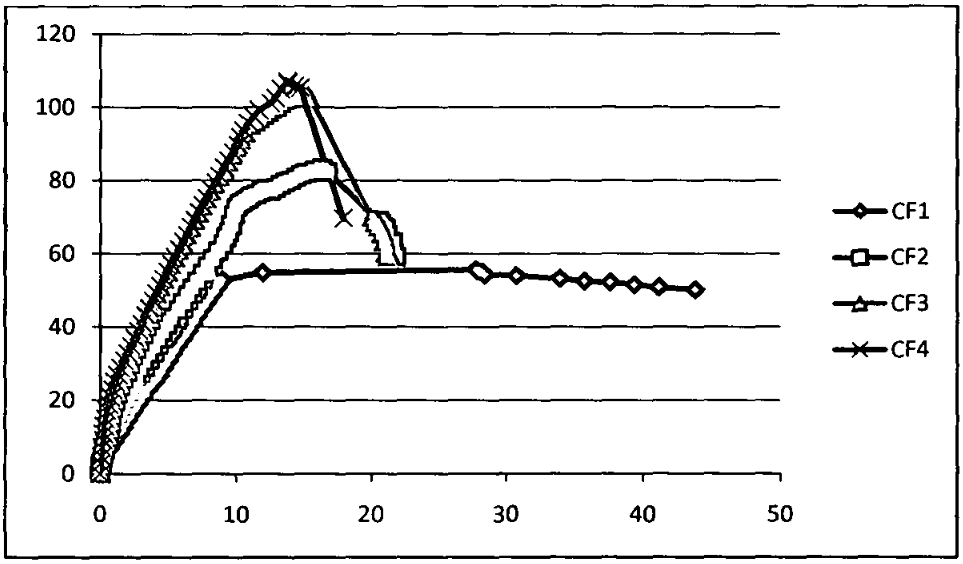

Figure 4.6 Load-deflection relationship for the CF-series

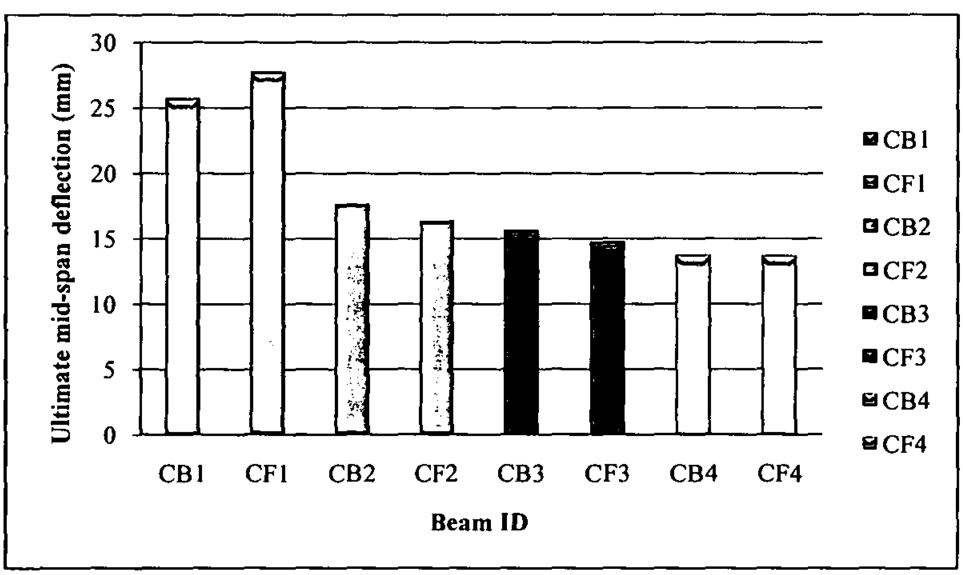

Figure 4.7 Ultimate mid-span deflections for all tested beams

4.3.2 Ductility

The ductility of a beam can be defined as its ability to sustain inelastic deformation without loss in the load-carrying capacity, prior to failure. It is usually calculated as the ratio of curvature, deflection or rotation at ultimate to yielding of steel [Almusallam and AlSalloum, (2001)], as given in equation 4.1.

$$\mu_D = \left(\frac{\Delta u}{\Delta y}\right) \qquad 4.1$$

Where:

μ_D: Ductility index

Δ_u: Mid-span deflection at ultimate load (mm), and

Δ_y: Mid-span deflection at yield load (mm).

The ductility index in this study was obtained based on the ultimate deflection obtained from the test and the calculated yield deflection and it was defined as the mid-span deflection, at ultimate load divided by the mid-span deflection at the point when the steel starts yielding. From the load-deflection curves in Figure 4.5 and 4.6, it can be seen that all CFRP-strengthened beams performed significantly better than the control beams with respect to load-carrying capacity. However, the observed increase in the strength was associated with the reduction in the deflection capacity of the respective beams.

The ductility indices for all tested beams are shown in Table 4.4. The values of the ductility indices for the CFRP-strengthened beams ranged from 2.7-3.5. The strengthened beams exhibited lower values as compared with the control specimens CB1 and CF1 of

4.6 and 5.0 ductility indices. The low ductility of strengthened beams indicates that the addition of the CFRP strengthening system reduced the deforming ability at the ultimate stage of loading. The reduction in ductility for the strengthened beams in reference to the control specimens is not considered to be significant. Therefore, all the strengthened beams were shown to have adequate ductility, which ensured that their failure mode was of a ductile nature.

Table 4.4 Ductility indices of the tested beams

Beam ID	Yield deflection (mm)[a] (Δ_y)	Ultimate deflection (mm)[b] (Δ_u)	Ductility index $\left(\frac{\Delta_u}{\Delta_y}\right)$
CB1	5.6	25.76	4.6
CF1	5.5	27.77	5.0
CB2	5.04	17.68	3.5
CF2	5.0	16.38	3.3
CB3	5.04	15.66	3.1
CF3	5.0	14.82	3.0
CB4	5.04	13.775	2.7
CF4	5.0	13.755	2.8

a: calculated from equation 3.15

b: experimental

4.3.3 Mode of Failure, Crack Pattern and Failure Load

All the strengthened beams exhibited higher load-carrying capacity when compared to the unstrengthened control beams. The ultimate failure loads for all beams are shown in Figure 4.8. The experimental results at ultimate level and the increase percentage at this level are shown in Table 4.3. Failure mode and crack pattern were physically observed during flexural testing of all beams. Cracks that appeared were marked for their extent and configuration and numbered according to the sequence of their appearance. Failure mode and crack pattern are discussed in the following section by considering the different CFRP length and concrete type.

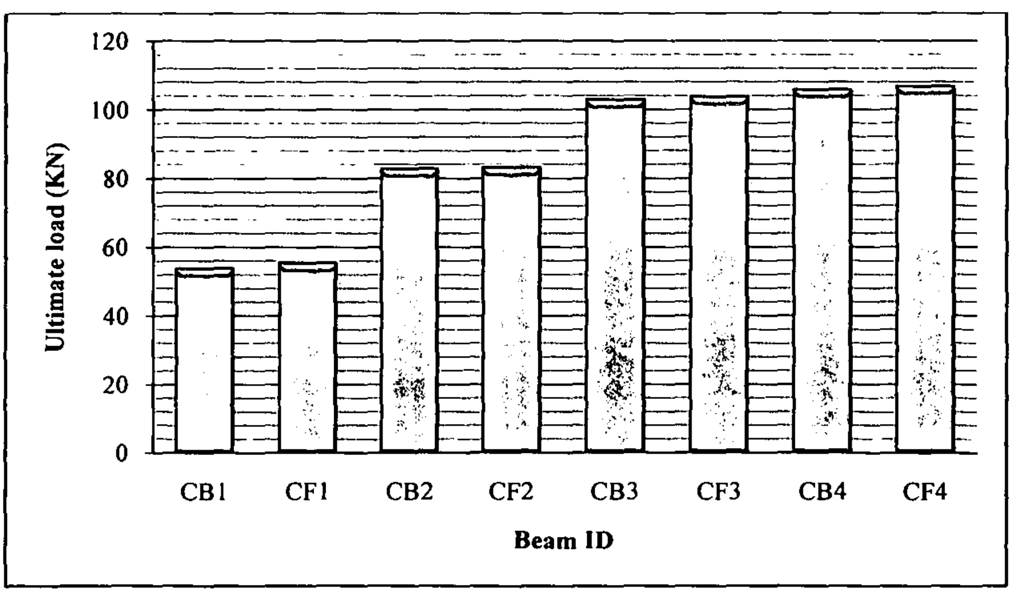

Figure 4.8 Ultimate loads for the tested beams

4.3.3.1 Effects of CFRP Plate Length

4.3.3.1.1 CB-series

Beam CB1 is the control specimen for CB-series and was made of ordinary concrete. It was not strengthened using CFRP. The beam exhibited small cracks at and around the loaded point and failed in flexure by crushing of concrete in compression zone; the failure was in a ductile manner, which can be attributed to the heavy shear reinforcement of the beam. The cracks started at the tension sides and increased in width and length with the applied load. The failure load for CB1 was 54.07 KN.

Beam CB2 which was strengthened with CFRP strip of 1333 mm length, i.e. at 67% of the total length, exhibited flexural cracks along the bending zone and snap sound was heard before the CFRP plate debonding, the failure mode for this beam was a hybrid of flexural and interfacial debonding of the CFRP plate. The ultimate load for CB2 was 83.03 KN which was 53.6% higher than unstrengthened control beam CB1.

Beams CB3 was strengthened with CFRP plates of length 1667 mm, i.e. at 83% of the total span length. This beam failed in the same manner as beam CB2 but with higher load-carrying capacity. The ultimate load for CB3 was 103.12 KN. The increase in the strength was 90.7% higher than the control specimen (CB1) and 24.2% over CB2.

Beams CB4 was strengthened with CFRP plate of length 2000 mm over the full span. Beam CB4 yielded the highest ultimate loads over all the strengthened beams. The failure mode for this beam was a hybrid mode of flexural and plate-end debonding. The ultimate load for CB4 was 106.06 KN, which was 96% higher than CB1, 27.7% higher than CB2 and 2.9% higher than CB3. The increase in the strength of CB4 over CB3 was 2.9%

which clarify the insignificant increase in the strength resulted by extending the length of the CFRP plate length from 83% to 100% of the total span length.

4.3.3.1.2 CF-series

Beam CF1 which contain 30% fly ash content was kept as the control specimen for the CF-series. It failed in a similar manner to CB1 i.e. in flexure by concrete crushing at the compression face with ultimate failure load of 55.56 KN. Beams CF2, CF3 and CF4 were strengthened with 67%, 83% and 100% of the full span length respectively, and they failed in the same manner (flexural failure and plate-end debonding) as their respective control specimens (normal concrete) CB2, CB3 and CB4 respectively. The failure loads for CF2, CF3 and CF4 were 83.4 KN, 103.89 KN and 106.99 KN which represents 50%, 87% and 92.6% higher than their control CF1.

It can be noticed that increasing the length of the CFRP plate increased the failure load and consequently increased the stiffness of the strengthened member. All the strengthened beams failed in similar way, flexure with plate-end debonding. The debonding failure can be attributed to the fact that flexural cracks formed in the constant moment region, as the load increased, the bond between the CFRP plate and concrete started to fracture at a certain load level, and the failure propagated towards the shear span until most parts of CFRP plate detached from the concrete beam. It can be seen that the bond between the CFRP plate and the concrete beam was not strong enough to ensure the rupture of the CFRP plate. There were widely spaced cracks in the non-strengthened control specimens CB1 and CF1. However; the cracks were narrower at relative close spacing in the strengthened beams. This shows the enhanced concrete confinement due to the CFRP strengthening. The crack patterns and the failure modes of all tested beams are shown in Figures 4.9-4.16.

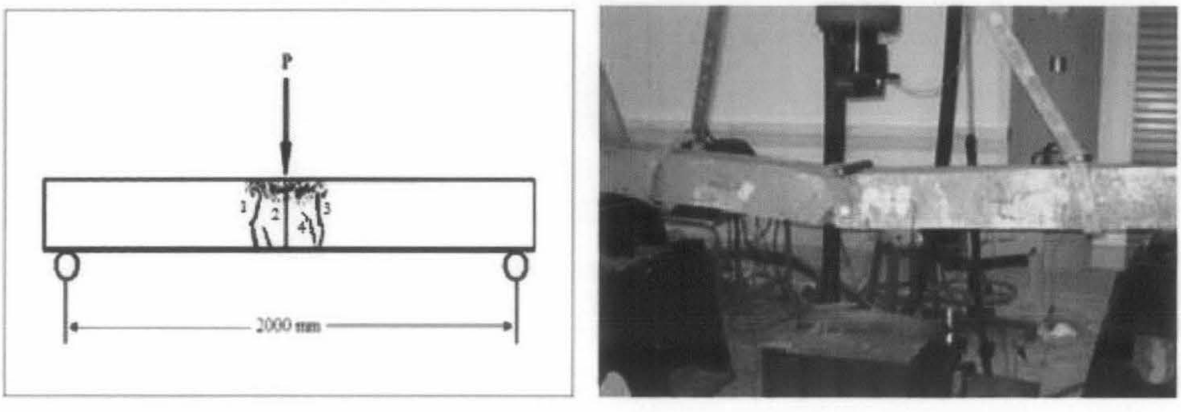

Figure 4.9 Crack patterns and failure mode of beam CB1

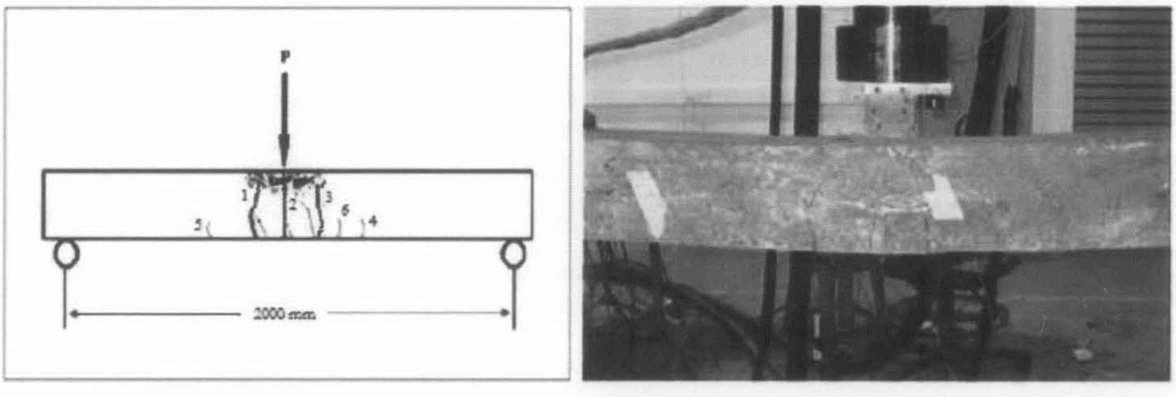

Figure 4.10 Crack patterns and failure mode of beam CF1

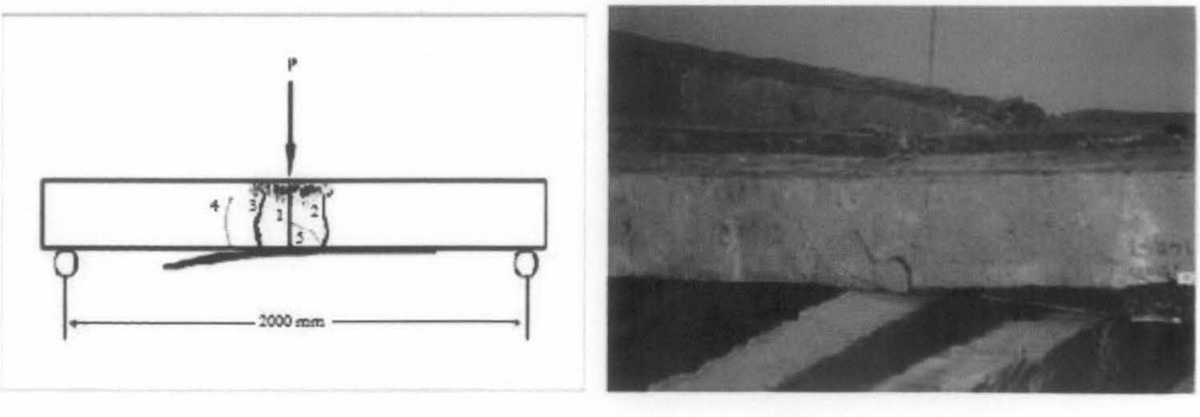

Figure 4.11 Crack patterns and failure mode of beam CB2

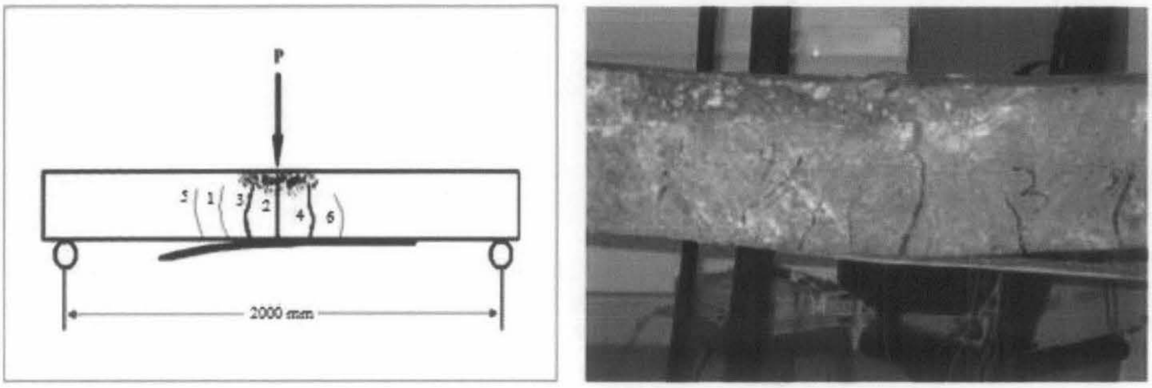

Figure 4.12 Crack patterns and failure mode of beam CF2

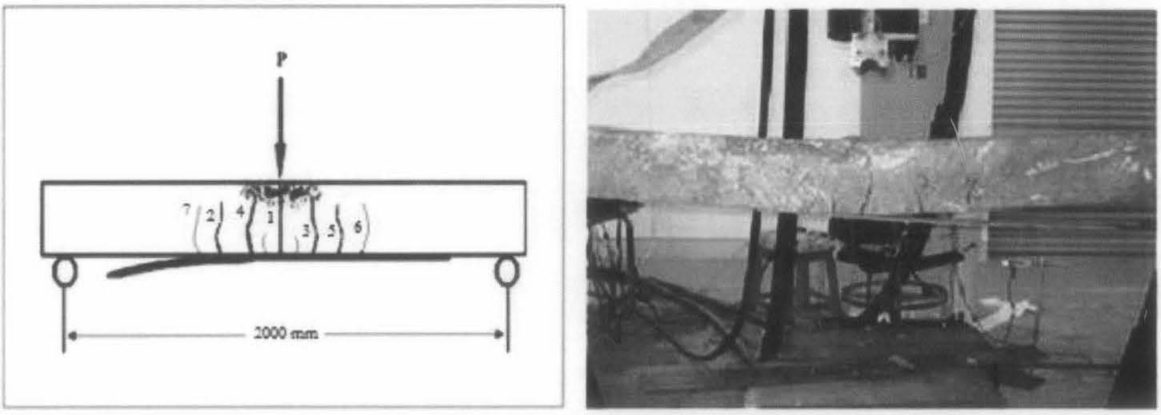

Figure 4.13 Crack patterns and failure mode of beam CB3

Figure 4.14 Crack patterns and failure mode of beam CF3

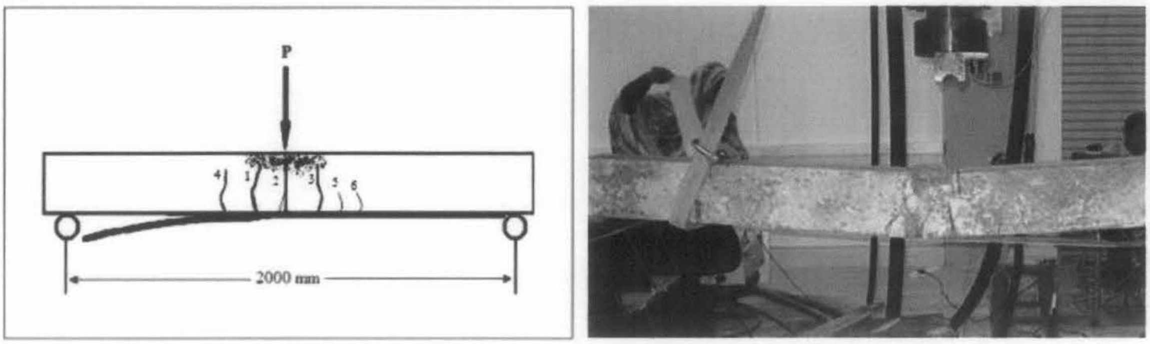

Figure 4.15 Crack patterns and failure mode of beam CB4

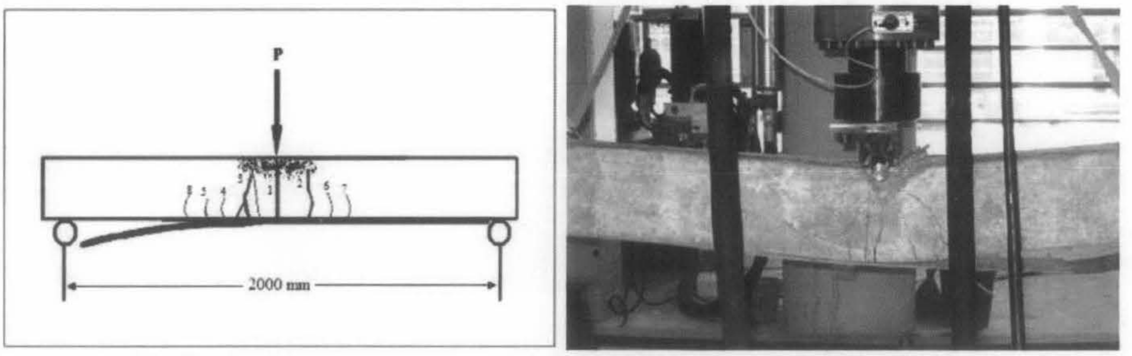

Figure 4.16 Crack patterns and failure mode of beam CF4

It can be seen that all CFRP-strengthened beams performed significantly better than the control beams with respect to the load-carrying capacity. However the observed strength increases were associated with reductions in the deflection capacity of the respective beams. It can also be observed that increasing the CFRP plate length ratio from 67% to 83% increased the load-carting capacity by about 27%, while by increasing the CFRP plate length from 83% to 100%, the load-carrying capacity was increased by about 2.9%. It can be seen that when the reinforcement goes from 83% to 100%, no apparent increase in the strength can be achieved. The reason behind the fact that 83% of the length was found to be the optimum length of the CFRP plate for flexural strengthening, can be attributed to the high compressive strength of the strengthened beams (69 MPa) which was effective in increasing the bond strength between the adhesive/concrete interface.

4.3.4 Experimental Ultimate Loads and Deflections versus Theoretical Results

The predicted results for the tested beams in terms of ultimate load and deflection were calculated based on equation 3.10 and 3.14 respectively [refer to Appendix B] and compared with the experimental results are graphically shown in Figures 4.17, 4.18 and 4.19, 4.20 respectively. These Figures show that the theoretical results calculated based on published data were correlated very well with the experimental results.

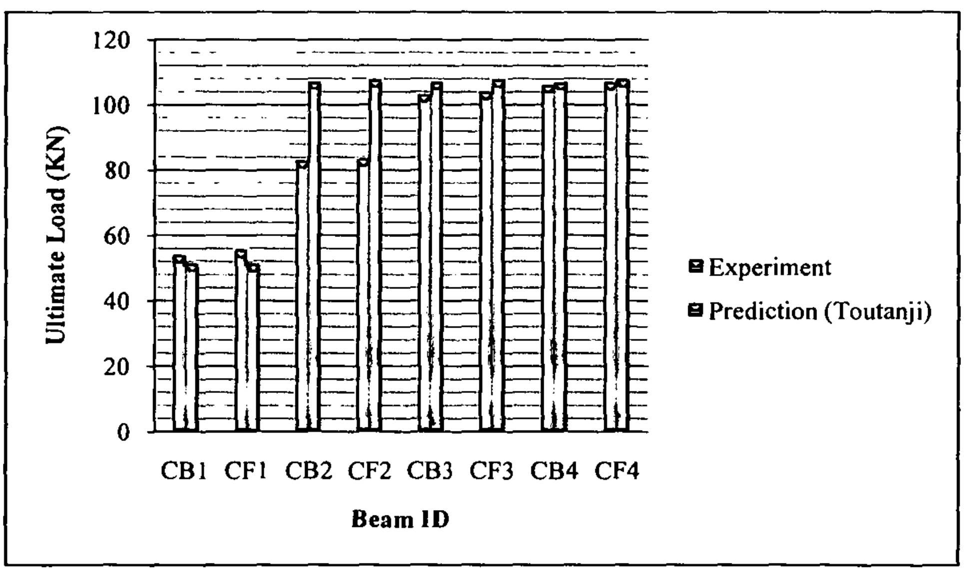

Figure 4.17 Comparison between the experimental and theoretical ultimate Load from published data

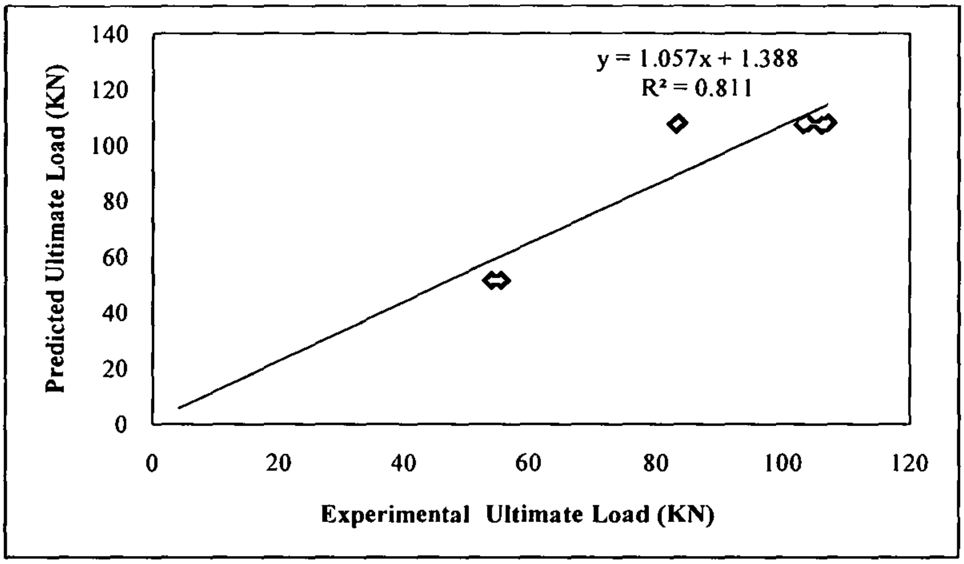

Figure 4.18 Experimental ultimate load versus prediction by Published data

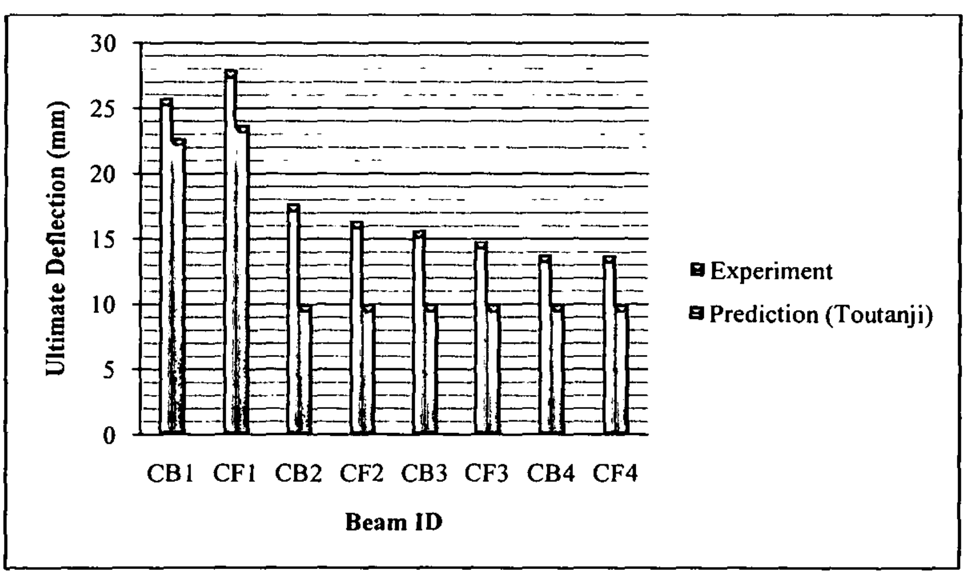

Figure 4.19 Comparison between the experimental and theoretical ultimate deflection from published data

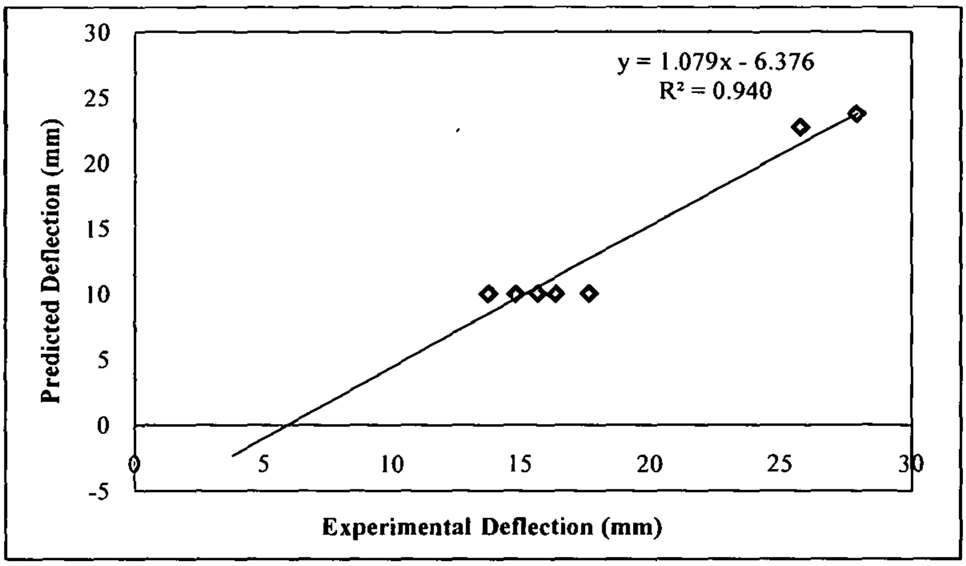

Figure 4.20 Experimental ultimate deflection versus prediction by published data

The ultimate loads were also calculated based on ACI 440 using equation 3.21 and 3.22 and compared with that obtained from the tests. Comparison between the test results and the theoretical results calculated from both published data and ACI code are presented in Table 4.2 and shown in Figures 4.21 and 4.22.

Figure 4.22 shows that the ACI 440.2R.02 under-estimates the experimental values of the ultimate load capacity carried by the CFRP-strengthened beams.

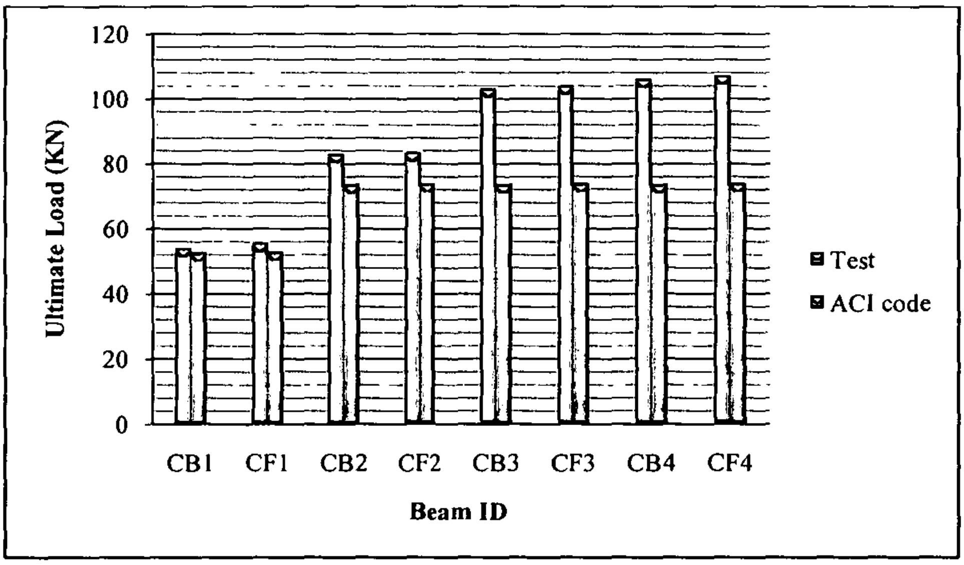

Figure 4.21 Comparison between the experimental and theoretical ultimate Load from ACI code

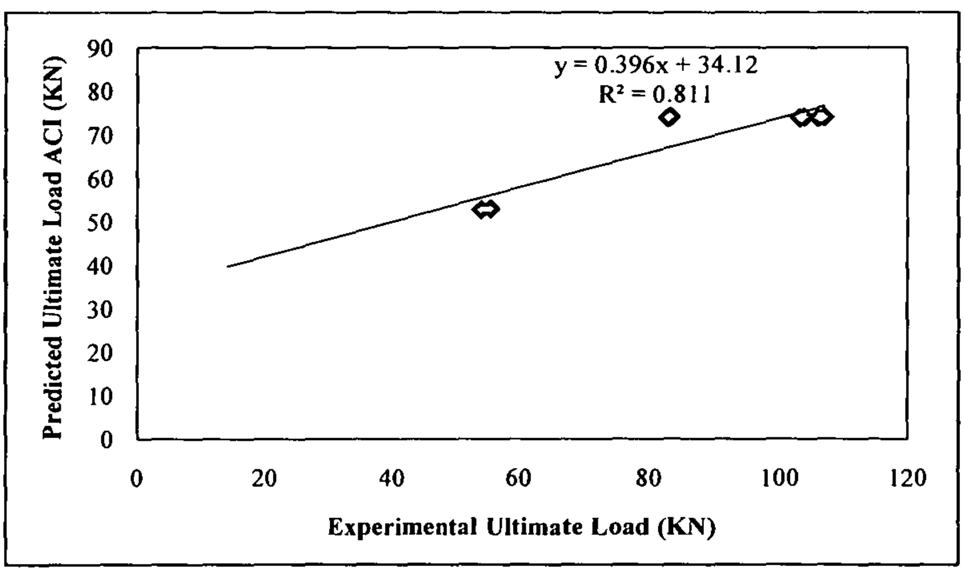

Figure 4.22 Experimental ultimate load versus prediction by ACI code

4.3.5 Verification of the Results with published data

Some previous studies have been done on the flexural behaviour of reinforced concrete beams externally strengthened with CFRP composites. For the purpose of comparison, the studies in which the researchers used the CFRP for flexural strengthening where the CFRP plate/sheet of length equal to the full span length of the beam were chosen. From the present study the beams CB3 and CB4 which were strengthened with a plate length equal to 83% and 100%, respectively, of the total span length were taken (area of interest). The increase percentage in the strength of each beam over the control was compared with the corresponding percentages from the previous studies. The ductility index from the present study was also compared to the ductility indices obtained from the previous studies. Beam dimensions and concrete and CFRP properties for the beams selected for comparison are summarized in Table 4.5. The selected beams are described as follows:

Benjeddou *et al.*, (2007) tested eight reinforced concrete beams while investigating the behaviour of damaged reinforced concrete beams (with different damage degrees) repaired by bonding of carbon fibre reinforced polymer laminates. The beam RB1 which was directly strengthened with 0% damage degree (not damaged) was taken for the comparison. In research by Toutanji *et al.*, (2006), three, four, five and six layers of carbon fibre were used to strengthen reinforced concrete beams. Beam 6L-1 was taken for comparison. Shahawy *et al.*, (1996) presents the test results of four reinforced rectangular concrete beams externally strengthened in flexure with bonded CFRP laminates. Beams S6-PRE2, and S6-PRE5 were taken for comparison. A study by Spadea *et al.*, (1998), in which three beams (A3.1, A3.2 and A3.3) bonded with CFRP plates with and without end anchorage. Beams A3.2 and A3.3 were selected for the comparison purpose.

As it can be seen from Figure 4.23; comparisons have shown that 83% of the total span length can achieve either similar or higher ultimate loads than that obtained by using 100% (full span length); which justify the use of this length as the optimum length of CFRP composite.

The ductility ratio of the strengthened beam can be defined as its ductility index divided by the ductility index of the control beam. The ductility ratios of the CFRP-strengthened beams were compared with the corresponding results obtained by Toutanji *et al*., (2006) and Spadea *et al*., (1998) as shown in Figure 4.24. From figure 4.24, the 83% of the total length of CFRP plate gained adequate ductility of 0.67 compared to the other researcher's ratios of 0.32, 0.64 and 0.54. Also the ductility ratio that gained by 100% plate length (0.58) was lower than that gained by 83% plate length (0.67) which explain very well this 83% of the span length can be considered as the optimum length in terms of load-carrying capacity and ductility.

Table 4.5 Beam test set-up from other studies

Author/Beam ID	Beam dimensions (mm)					Concrete properties		Tension reinforcement				FRP properties		
	b	h	d	L	a	E_c (GPa)	f'_c/f_{cu} (MPa)	E_s (GPa)	f_y (MPa)	A_s (mm^2)	A'_s (mm^2)	E_f (GPa)	ε_{fu} (%)	$A_f(mm^2) = n \times t_f \times b_f$
Benjeddou *et al.*, (2007)/RB1	120	150	110	1800	600	30	21	200	400	157	101	165	1.7	1 × 1.2 × 100
Toutanji *et al.*, (2006)/6L-1	108	160	106	1526	560	33.1	49	200	427	142	56	n/a	n/a	6 × 0.165 × 102
Shahawy *et al.*, (1996)/S6-PRE2	203	305	251	2440	1067	30.4	41.4	234	469	265	0	141.3	1.95	2 × 0.17018 × 300
Shahawy *et al.*, (1996)/S6-PRE5	203	305	251	2440	1067	30.4	41.4	234	469	265	0	141.3	1.95	3 × 0.17018 × 300
Spadea *et al.*, (1998)/A3.2	140	300	263	4800	1800	26	30	217.5	435	402	402	152	1.51	1 × 1.2 × 80
Spadea *et al.*, (1998)/A3.3	140	300	263	4800	1800	26	30	217.5	435	402	402	152	1.51	1 × 1.2 × 80
Present study, CFRP length 83%	150	200	161	2000	1000	34	67.5	230	460	402	0	150	1.4	1 × 1.2 × 100
Present study, CFRP length 100%	150	200	161	2000	1000	34	67.5	230	460	402	0	150	1.4	1 × 1.2 × 100

n/a: Not available

0: The beam is designed as singly reinforced section

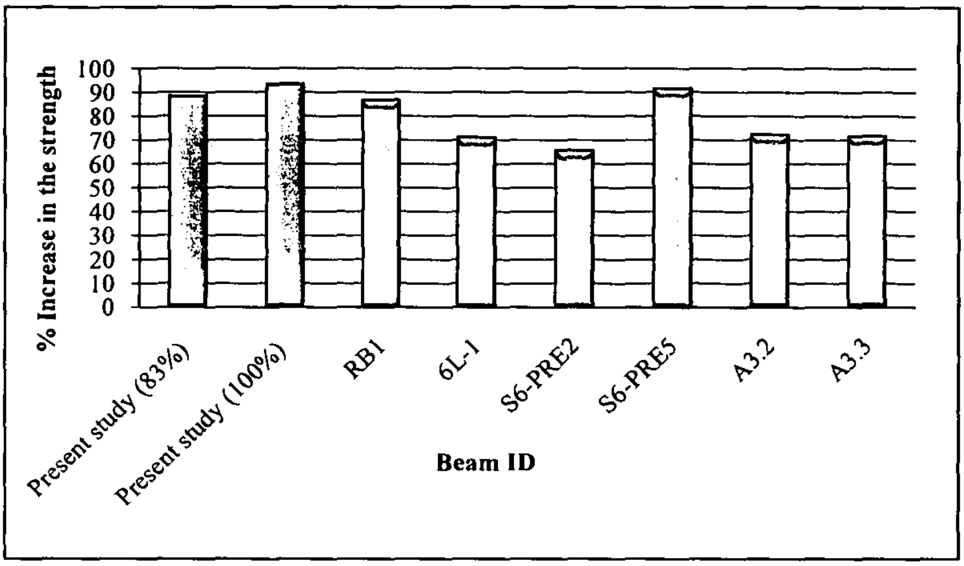

Figure 4.23 Percentage increase in the ultimate load between the current study and previous studies

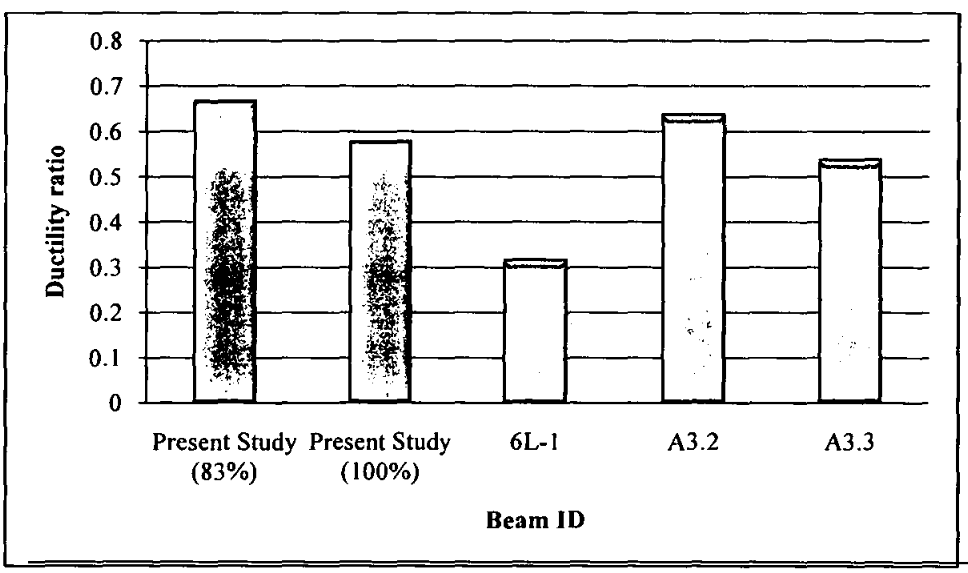

Figure 4.24 Ductility ratios of the present study and other researchers

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusions

This study was done in two major parts in order to align with the designed objectives. The first part was aimed to determine the concrete mix proportion for a target 28-day compressive strength of 50-60 MPa that incorporate the most optimum cement content. The second part was mainly aimed to determine the optimum length of CFRP strips for maximum flexural strength of RC beams made of normal and fly ash blended cement concrete. The experimental results must be supported by the theoretical analysis using existing models. Following were the important conclusions drawn:

1. The lowest cement content of 228 kg/m^3 with a fly ash content of 152 kg/m^3 provided 28-day compressive strength of 52 MPa. The strength development process did not stop or slow down after 28 days. At the age of 90 days the compressive strength was 69 MPa, about 30% higher than that of 28-days strength.
2. At 28 days, the highest compressive strength was obtained by the replacement of cement with 20% fly ash content, where the cement plus fly ash content was 380 kg/m^3. The 28-day compressive strength of this mix was 68 MPa that was further increased by 28% to 84 MPa at 90 days.
3. The higher compressive strengths were achieved by keeping the w/b ratio to 0.4.
4. The optimum CFRP plate length was obtained as 83% of the span length at the beam soffit for the flexural strengthening. Beam made of 30% fly ash content exhibited a failure load of 104 KN and a maximum deflection of 15 mm. Similarly the control beam failed at 103 KN and 16 mm deflection.

5. Beams strengthened with 83% CFRP length achieved an ductility index of 3.1 as compared to that of the control beams, which was 4.6; hence a ductility ratio obtained as 0.67 is considered adequate as discussed in the literature.
6. Results of ultimate load and deflection of 30% fly ash blended cement concrete beams were almost identical with respect to the control beams.
7. Experimental results of loads and deflections correlated well with the theoretically calculated loads and deflections.

5.2 Recommendations for Future Work

- Anchorage system must be provided to prevent debonding failure.
- The effects of different widths and thicknesses of CFRP plates for both flexural and shear strengthening should be investigated and the optimum width and thickness of CFRP plates for flexural strengthening need to be established.

Appendix A

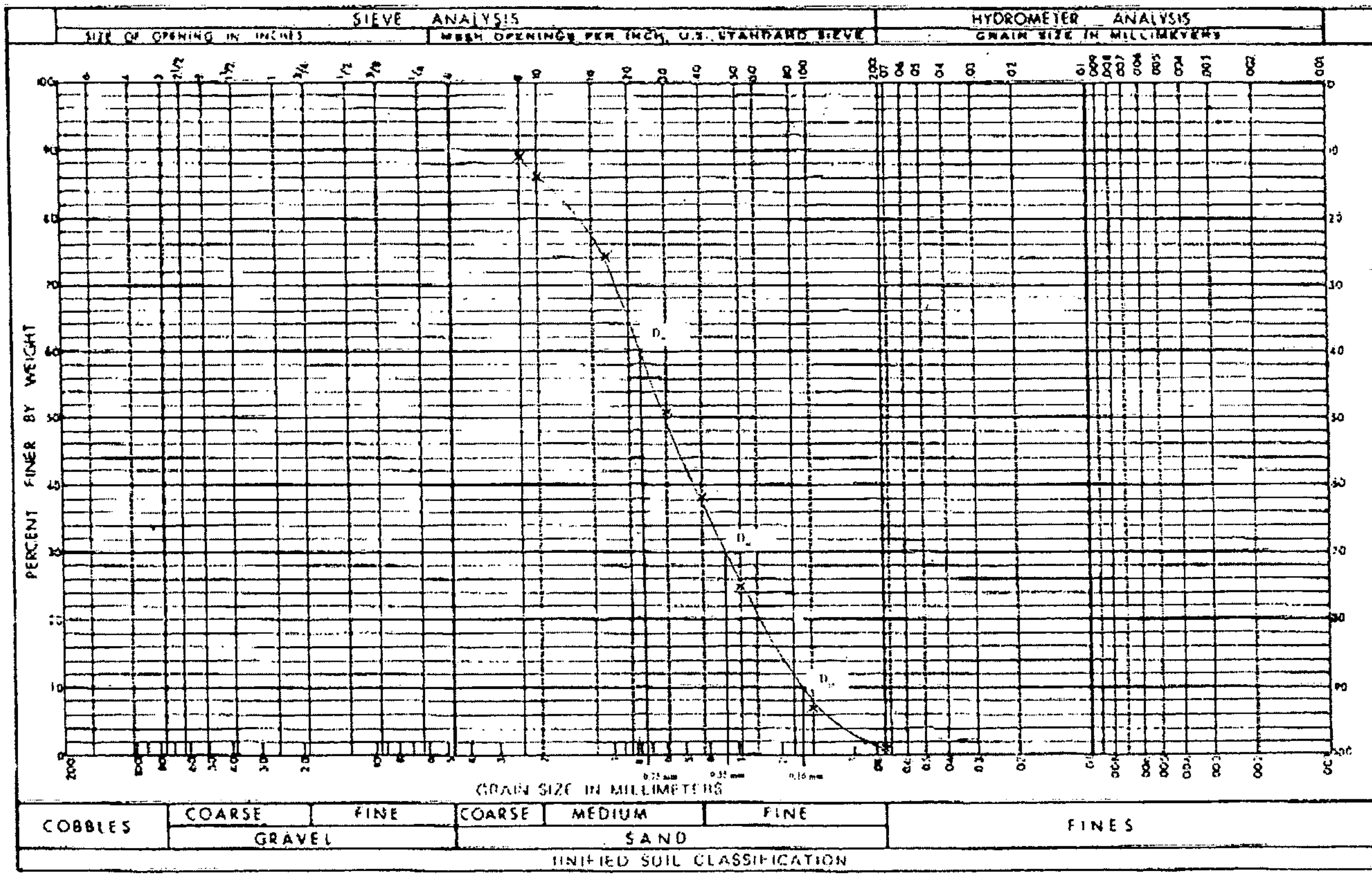

Figure A1 Particle Size Distribution Curve for Fine Aggregate

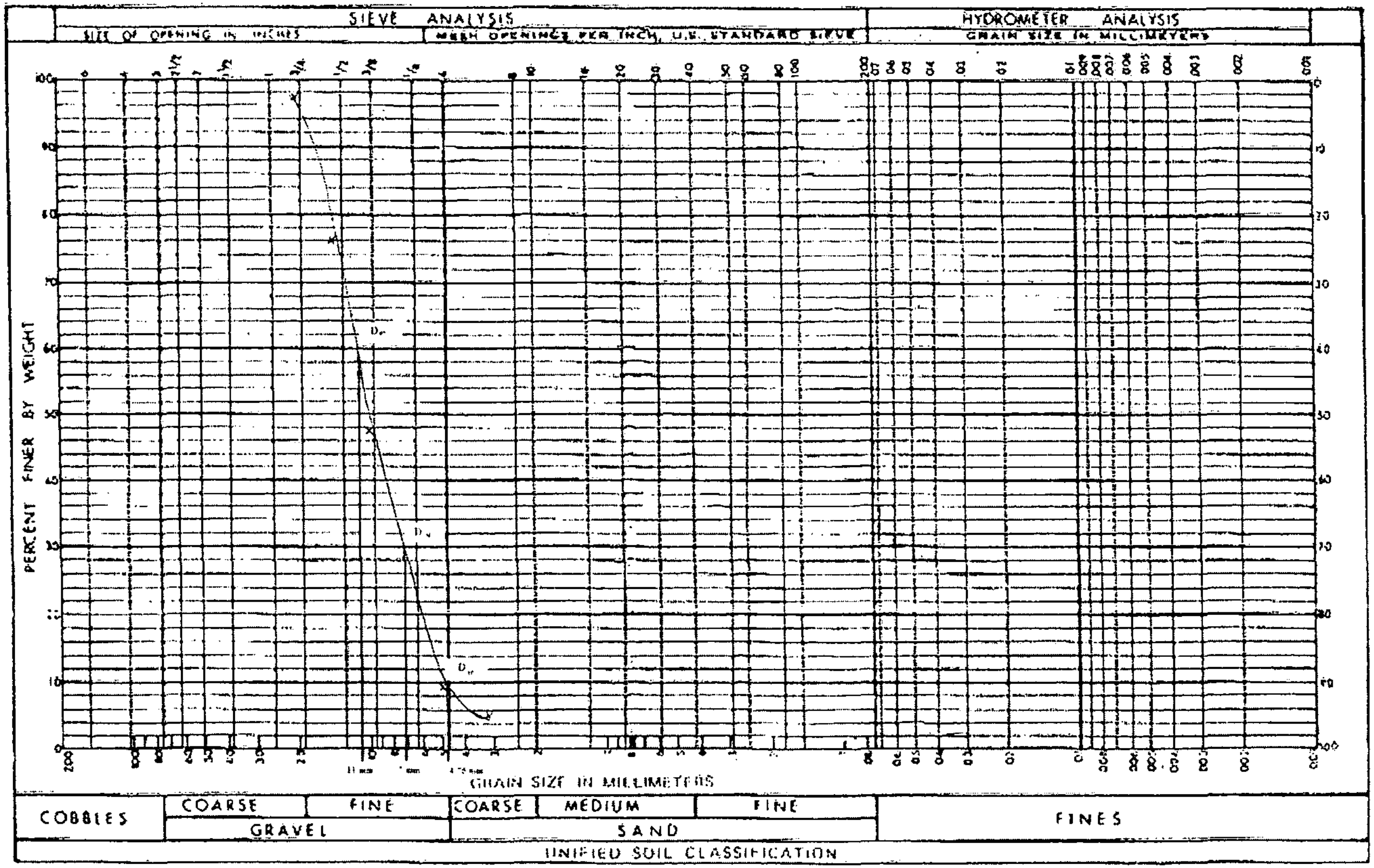

Figure A2 Particle Size Distribution Curve for Coarse Aggregate

Appendix B

Calculated Parameters

Beam CB1

$c = 43\ mm$

$\varepsilon_c\ at\ yield\ level = 0.00073$

$\emptyset\ at\ yield\ level = 1.69 \times 10^{-5}\ (1/mm)$

$\Delta_y = 5.6\ mm$

$\emptyset\ at\ ultimate\ level = 6.98 \times 10^{-5}\ (1/mm)$

$\Delta_u = 23.2\ mm$

$M_u = 25.7\ KN.m$

Beams CF1

$c = 41\ mm$

$\varepsilon_c\ at\ yield\ level = 0.00068$

$\emptyset\ at\ yield\ level = 1.66 \times 10^{-5}\ (1/mm)$

$\Delta_y = 5.5\ mm$

$\emptyset\ at\ ultimate\ level = 7.32 \times 10^{-5}\ (1/mm)$

$\Delta_u = 24.3\ mm$

$M_u = 25.7\ KN.m$

Beams CB2, CB3 and CB4

$c = 48\, mm$, $I_{cr} = 52.49 \times 10^6 mm^4$

Yield Level

$\varepsilon_c = 0.00085$

$\varepsilon_f = 0.0027$

$k_1 = 0.36$

$k_2 = 0.65$

$M_y = 26.97\, KN.m$

$\Delta_y = 5.04\, mm$

Ultimate Level

$\varepsilon_{ff} = 0.0083$

$k_1 = 0.75$

$k_2 = 0.58$

$M_u = 53.5\, KN.m$

$\Delta_u = 10\, mm$

Beams CF2, CF3 and CF4

$c = 46.7\ mm$, $I_{cr} = 53.0 \times 10^6 mm^4$

Yield Level

$\varepsilon_c = 0.00082$

$\varepsilon_f = 0.0027$

$k_1 = 0.35$

$k_2 = 0.65$

$M_y = 27.02\ KN.m$

$\Delta_y = 5.0\ mm$

Ultimate Level

$\varepsilon_{ff} = 0.0083$

$k_1 = 0.75$

$k_2 = 0.58$

$M_u = 53.85\ KN.m$

$\Delta_u = 9.96\ mm$

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